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ONE-DIMENSIONAL WAVE-PROPAGATION ANALYSIS, NEWBURGH LOCKS AND D--ETC(U)
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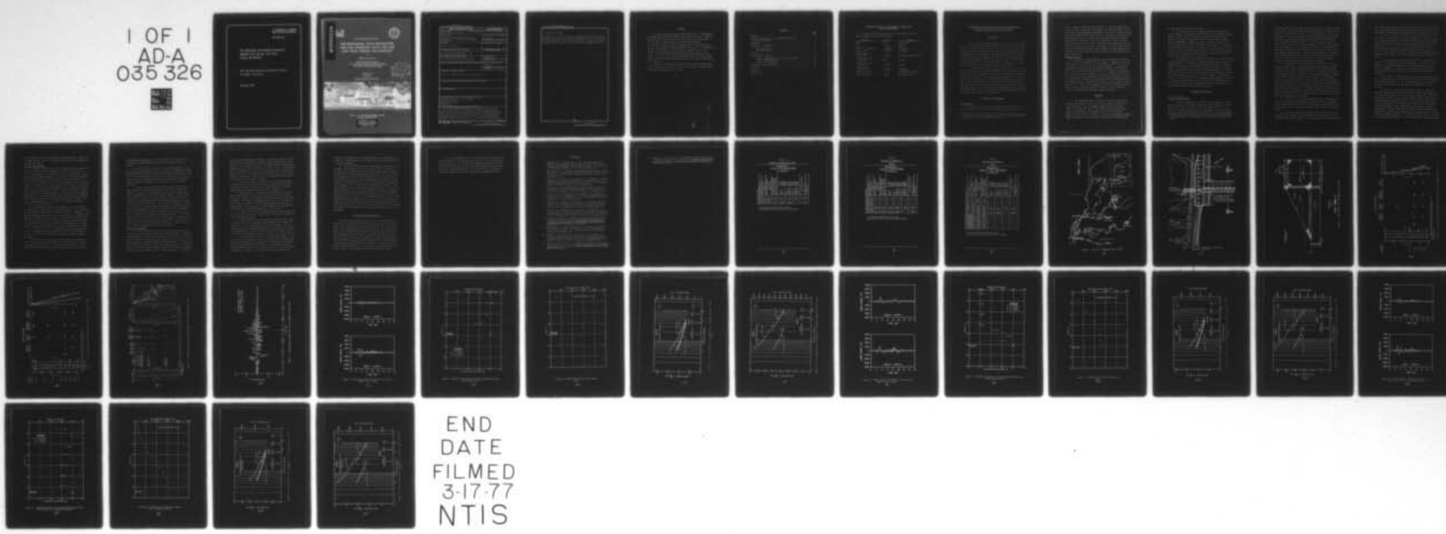

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ONE-DIMENSIONAL WAVE-PROPAGATION ANALYSIS NEWBURGH LOCKS AND DAM, OHIO RIVER INDIANA AND KENTUCKY

ARMY ENGINEER WATERWAYS EXPERIMENT STATION
VICKSBURG, MISSISSIPPI

DECEMBER 1976

MISCELLANEOUS PAPER S-76-25

ONE-DIMENSIONAL WAVE-PROPAGATION ANALYSIS, NEWBURGH LOCKS AND DAM OHIO RIVER, INDIANA AND KENTUCKY

by

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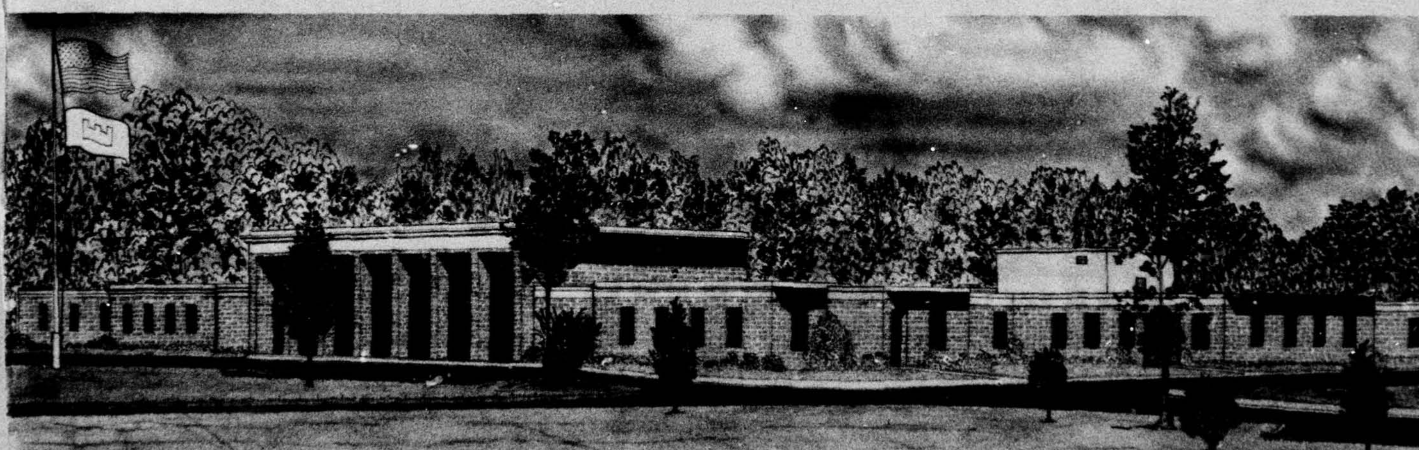
December 1976
Final Report



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The site of the Newburgh Locks and Dam, located on the Ohio River 8 miles upstream from the confluence of the Ohio and Green Rivers near Evansville, Indiana, was investigated with regard to liquefaction potential. Cyclic triaxial compression tests were performed on remolded specimens of the foundation material at various densities. A one-dimensional wave-propagation analysis was performed for two typical soil profiles under the fixed-weir portion of the structure. As a result of this analysis, it is

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20. ABSTRACT (Continued).

recommended that a study be initiated to determine the cost and benefits of: (a) removing the top 20 ft of sand below and downstream of the toe of the rock berm and replacing this material with nonliquefiable material; (b) extending the existing rock berm in the downstream direction. It is believed that such modifications might render the structure stable under earthquake excitation.

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Preface

The study reported herein was performed by the U. S. Army Engineer Waterways Experiment Station (WES) at the request of the U. S. Army Engineer District, Louisville, and was authorized in Intra-Army Order No. DC-B-76-11, dated 19 August 1975, and Intra-Army Order No. DC-B-76-1, change 1, dated 12 September 1975. The study was conducted during the period September-November 1975 by Dr. W. F. Marcuson III and Mr. G. R. Skoglund, under the general supervision of Dr. F. G. McLean, Chief, Earthquake Engineering and Vibrations Division, and Messrs. R. G. Ahlvin and J. P. Sale, Assistant Chief and Chief, respectively, of the Soils and Pavements Laboratory. This report was prepared by Dr. Marcuson.

Directors of WES during the conduct of this study and the preparation of this report were COL G. H. Hilt, CE, and COL J. L. Cannon, CE. Technical Director was Mr. F. R. Brown.

Contents

Conversion Factors, U. S. Customary to Metric (SI) Units of Measurement

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	25.4	millimetres
feet	0.3048	metres
miles (U. S. statute)	1.609344	kilometres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (force) per square inch	6894.757	pascals
pounds (force) per square foot	47.88026	pascals
kips (force) per square foot	47.88026	kilopascals
inches per second	25.4	millimetres per second
feet per second	0.3048	metres per second

ONE-DIMENSIONAL WAVE-PROPAGATION ANALYSIS, NEWBURGH LOCKS AND DAM, OHIO RIVER, INDIANA AND KENTUCKY

Background

1. The U. S. Army Engineer District, Louisville (LED), has responsibility for designing and constructing a lock and dam structure on the Ohio River 8 miles* upstream from the confluence of the Ohio and Green Rivers near Evansville, Indiana (see Figure 1). In December 1972, LED requested the U. S. Army Engineer Waterways Experiment Station (WES) to conduct a laboratory investigation to evaluate the dynamic response of a cohesionless foundation material under the structure. Cyclic triaxial tests were performed on remolded specimens of this material at various relative densities. The results are reported in Reference 1.

2. In order to gain insight into the dynamic response of the foundation material under the fixed-weir portion of this structure and the material in the left abutment, LED requested in September 1975 that WES conduct one-dimensional (1-D) wave-propagation analyses for several soil profiles at the structure. The results of these 1-D analyses are presented herein. These analyses are considered preliminary in the sense that only the minimum field investigation necessary for dynamic analyses has been conducted. For these analyses, WES assumed that the geological and seismological factors influencing the design earthquake were exactly the same as those reported for Patoka Dam.[2]

Definition of Parameters

Soil profiles

3. Figure 2 is a plan view of a portion of the fixed weir of the lock and dam structure. Section C-C through cell 13 was chosen as

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is given on page 3.

typical. Figure 3 shows the cross section C-C. Two soil columns were chosen from this cross section for analyses. As shown in Figure 3, profile A was located immediately downstream of the toe of the rock berm and profile B immediately downstream of cell 13. The third case selected for analysis was profile C, located in the left abutment near boring D-500 (see Figure 2). Figures 4, 5 and 6 are profiles A, B, and C, respectively, as analyzed in this study. Throughout this report these will be referred to as Cases A, B, and C, respectively. Soil properties such as density and shear wave velocity shown in Figures 4-6 were estimated on the basis of published data[3] and are consistent with WES experience with similar materials.

Earthquake motions

4. In order to develop expeditiously the geological and seismological factors, the Newburgh site was assumed, for geologic and seismologic purposes, to be the same as the Patoka Damsite.[2,4] One bedrock earthquake motion was used for the Newburgh analyses. The characteristics of this event were identical with those described in References 2 and 4. The epicenter was assumed to occur at a distance of 6 km from the site, and the Richter magnitude of the event was assumed to be 6.25. To represent this earthquake, an acceleration-time history developed for the dynamic analysis of Lopez Dam[5] was scaled to have a maximum acceleration of 0.4 g. This scaled earthquake motion has a maximum velocity of 21.6 in./sec, a maximum displacement of 10.6 in., and a duration of 16 sec. The unscaled accelerogram is shown in Figure 7.

Analysis

5. Soil parameters shown in Figures 4-6 and the acceleration-time history, Figure 7, scaled to 0.4 g, were used as input for 1-D response analyses. The 1-D analysis assumes infinite horizontal layers in all directions and no shear stresses on horizontal planes before the earthquake. The computer code used for this study, SHAKE,[6] provides a Fourier solution to the 1-D wave-propagation problem. SHAKE was developed at the University of California at Berkeley under

the supervision of Professor H. B. Seed.

6. In these analyses, the scaled acceleration-time history was input at bedrock. The computer program treats the soil layers as having nonlinear elastic properties, using an equivalent linear procedure.[7] Using this procedure, the program iterates to obtain stress-strain-damping compatibility. Important parts of the computer output are: (a) the maximum shear stress at the center of each layer, (b) the maximum acceleration at the top of each layer, and (c) the shear stress versus time history at the center of each layer. These shear stress-time histories are compared with laboratory cyclic triaxial test results to evaluate the strain potential of each layer.

7. The laboratory test specimens are subjected to periodic uniform stresses, whereas a nonuniform acceleration-time history is used in the computer program. To compare the laboratory results with the computer output, it is necessary to determine the equivalent number of uniform stress cycles from the computer shear stress-time history. By "equivalent" it is meant that the soil responds in the same manner under either the computed stress-time history or the uniform periodic stress-time history. A modified version of the computer program, EQCYCLE,[8] was used to compute the average stress based on a number of equivalent cycles. This procedure incorporates the response of the soil materials into the process of determining the uniform stress values.

Presentation of Results

Case A - downstream of toe of rock-fill berm

8. Figure 4 shows the soil profile that was analyzed for Case A using the computer code SHAKE. The water level was assumed at el 342,* the top of the river bottom at el 285, and bedrock at el 247. The 38-ft soil column was divided into two sand layers. Based on Standard Penetration Test (SPT) data obtained by LED, the top 20 ft was assigned

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* All elevations (el) cited herein are in feet referred to mean sea level.

a relative density of 60 percent, and the bottom 18 ft a relative density of 80 percent. Each layer was further subdivided into two layers for the analysis. A shear wave velocity of 400 fps was assumed for the top two layers, and a shear wave velocity of 700 fps was assumed for the bottom two layers. Initial shear moduli of 620 and 1900 ksf were determined from the assumed shear wave velocities of the top and bottom layers, respectively. The initial modulus was assumed to be uniform with depth. An initial damping ratio of 5 percent was assumed. Since modulus and damping are functions of strain, the program iterates to obtain stress-strain-damping compatibility. On the last program iteration, after compatibility was obtained, shear stress-time histories were obtained for the tops of layers 2 and 4, corresponding to an average damping of 21 percent. These time histories are shown in Figure 8.

9. Maximum values of acceleration were obtained at the top of each of the four layers. These values are shown in Figure 9. Because the shear moduli (G) were estimated based on previous WES experience, they were varied by plus or minus 25 percent to determine the influence of the estimate on the calculations. The maximum accelerations produced in the profile by using 125 percent and 75 percent of the estimated moduli are also shown in Figure 9. The variation in maximum acceleration is on the order of plus or minus 10 percent, which is not considered significant. Using the estimated values of shear moduli, Figure 9 shows that the bedrock acceleration is initially deamplified from bedrock to a depth of 29 ft, after which the amplification ratio increases from 0.75 at the 29-ft depth to 1.1 at the ground surface. The peak acceleration, 0.44 g, occurs at the ground surface.

10. Figure 10 is a plot of maximum shear stress versus depth for Case A, as obtained from the computer program SHAKE for the estimated values. Maximum shear stress values are plotted at the midpoints of the corresponding layers. As expected, maximum shear stress increases with depth.

11. The shear stress-time histories shown in Figure 8 were used as input for the computer program EQCYCLE, as modified by WES, to compute the average shear stress for 3, 5, and 10 equivalent uniform cycles.

For 5 equivalent cycles, the average shear stress was found to be 57 percent of the maximum shear stress or 319 psf at the top of layer 2, and 60 percent of the maximum shear stress or 675 psf at the top of layer 4.

12. Figures 11 and 12 are plots of cyclic shear stress, τ , in psi and psf, versus number of cycles of loading, on a logarithmic scale, for the Newburgh material at approximately 60 and 80 percent relative density, respectively. These plots are considered representative of the material at el 275 (top of layer 2) and el 256 (top of layer 4). These figures were obtained by taking the plots of stress ratio, $\sigma_{dc}/2\sigma_c$, versus number of cycles of loading given in Reference 1, and multiplying the stress ratio by the appropriate effective overburden stress.

13. Recent research at WES and elsewhere[9,10] indicates that the response of reconstituted laboratory specimens is different from the response of undisturbed samples tested in the laboratory using cyclic triaxial equipment. On the basis of this research, the laboratory strengths were increased 65 percent to correct for the effects of remolding.

14. In order to correct the cyclic triaxial test data to equivalent in situ conditions, the cyclic triaxial stresses must be multiplied by the correction factor C_r .[11] For this analysis C_r was chosen as 0.60. These factors were included in the plots shown in Figures 11 and 12.

15. Using the criterion of five cycles, Figure 11 shows that shear stresses of 161, 170, and 216 psf will cause initial liquefaction, 10 percent peak-to-peak strain, and 20 percent peak-to-peak strain, respectively, in the top layer of sand. Figure 12 shows, using the five-cycle criterion, that shear stresses of 720 and 1095 psf will cause initial liquefaction and 10 percent peak-to-peak strain, respectively, in the bottom layer of sand. These values are tabulated in Table 1. If a factor of safety is defined as the shear strength of the material divided by the shear stress produced by the earthquake, then it is seen that the top layer of sand, i.e., the top 20 ft, has a factor of safety of 0.53 and the bottom layer, i.e., the bottom 18 ft, a factor of safety

of 1.62, based on a failure criterion of 10 percent strain. Thus, it is concluded that the top 20 ft of sand in Case A will fail if subjected to the design earthquake.

Case B - with rock berm

16. Figure 5 shows the soil profile that was analyzed for Case B. The top of the rock berm was assumed at el 349, the water level at el 342, and the base of the rock berm at el 285. The top layer of foundation sand was assumed between el 285 and 265; this sand was assumed to have a relative density of 60 percent and a shear wave velocity of 400 fps. The sand between el 265 and 247 was assumed to have a relative density of 80 percent and a shear wave velocity of 700 fps. These shear wave velocities yield initial shear moduli of 620 and 1900 ksf, respectively, and an initial damping ratio of 5 percent was assumed. As mentioned previously, the program iterates to obtain stress-strain-damping compatibility. On the last iteration, after compatibility was obtained, shear stress-time histories were obtained for the tops of layers 6 and 8 for an average damping of 13.3 percent. These shear stress-time histories are shown in Figure 13.

17. The maximum acceleration was obtained at the top of each of the eight layers. These values are shown in Figure 14. The estimated shear modulus was varied by plus or minus 25 percent for comparison. The maximum accelerations produced in the profile are shown in Figure 14 for each modulus assumption. The variation in maximum acceleration is less than plus or minus 10 percent. Figure 14 shows that with the estimate of shear modulus, the bedrock acceleration is deamplified in the founding material and remains essentially constant up through the rock berm.

18. Figure 15 is a plot of maximum shear stress versus depth for Case B. The maximum shear stress values are plotted at the center of each of the layers. As expected, the maximum shear stress increases with depth.

19. Shear stress-time histories shown in Figure 13 were used as input in the computer program EQCYCLE, as modified by WES, to compute the average shear stress for 3, 5, and 10 equivalent cycles. For five equivalent cycles, the average shear stress was found to be 52 percent

of the maximum shear stress, or 524 psf, at the top of layer 6, and 65 percent of the maximum shear stress, or 653 psf, at the top of layer 8.

20. Figures 16 and 17 are plots of cyclic shear stress, τ , in psi and psf, versus number of cycles of loading for the Newburgh material at approximately 60 and 80 percent relative density, respectively. These plots are for the material at el 275 (top of layer 6) and el 256 (top of layer 8) and were obtained by appropriately modifying the laboratory results presented in Reference 1, and were corrected as mentioned previously.

21. Figure 16 shows that under the criterion of five cycles, shear stresses of 1240, 1298, and 1658 psf will cause initial liquefaction, 10 percent peak-to-peak strain, and 20 percent peak-to-peak strain, respectively, in the top sand layer. Figure 17 shows that the criterion of five cycles gives shear stresses of 2430 and 3670 psf to cause initial liquefaction and 10 percent peak-to-peak strain, respectively, in the bottom sand layer. These values are tabulated in Table 2. If, as in Case A, a factor of safety is defined as the shear strength of the material divided by the shear stress produced by the earthquake, then it is seen that the top layer of sand, i.e., the top 20 ft, has a factor of safety of 2.47 and the bottom layer of sand, i.e., the bottom 18 ft, has a factor of safety of 5.63. These factors are based on a failure criterion of 10 percent peak-to-peak strain. Thus, it is concluded that the foundation material beneath the rock berm for Case B will not fail if subjected to the design earthquake.

Case C - left abutment

22. Figure 6 shows the soil profile that was analyzed for Case C. This profile was developed from the log of LED boring D-500. Ground surface was assumed at el 376. A description of the various soil layers is shown in Figure 6. The medium to fine sand shown from el 350 to el 317 was the material of primary interest during this analysis. Below el 317 was a medium dense, coarse to medium gravelly sand. It was not possible to analyze this layer because dynamic laboratory data were not available. The top of rock was assumed at el 248. This soil profile was subdivided

into 15 layers for purposes of analysis. Also shown in Figure 6 are the estimated unit weights and the estimated relative densities of the respective layers, along with a plot of overburden pressure versus depth. Immediately to the right of this plot is a plot of SPT N values versus depth. Superimposed on this plot are correlation curves developed by Gibbs and Holtz.[12] This plot was used to estimate the in situ relative density of the material. Estimates of the shear wave velocity and the shear modulus are also shown in Figure 6.

23. On the last program iteration, after stress-strain-damping compatibility was obtained, shear stress-time histories were obtained for the tops of layers 6 and 8 for an average damping of 20 percent. These shear stress-time histories are shown in Figure 18.

24. Maximum values of acceleration were obtained at the top of each of the 15 layers. These values are shown in Figure 19 compared with values obtained by varying the estimated modulus by plus and minus 25 percent. The variation in maximum acceleration is on the order of plus or minus 10 percent. Figure 19 shows that with the estimate of shear modulus, the acceleration is fairly uniform in the saturated soil in this profile and generally ranges from 0.32 to 0.41 g.

25. Figure 20 is a plot of maximum shear stress versus depth for Case C, plotted at the center of each layer. As expected, maximum shear stress increases with depth.

26. Using the shear stress-time histories shown in Figure 18 and a modified version of the computer program EQCYCLE, the average shear stress for 3, 5, and 10 equivalent cycles was calculated. For five equivalent cycles, the average shear stress was found to be 62 percent of the maximum shear stress, or 386 psf, at the top of layer 6 and 62 percent of the maximum shear stress, or 680 psf, at the top of layer 8.

27. Figures 21 and 22 are plots of cyclic shear stress, τ , in psi and psf, versus number of cycles of loading for the Newburgh material at approximately 60 and 80 percent relative density, respectively. These plots were obtained from Reference 1 and have been modified to represent depths of 36 ft (top of layer 6) and 52.5 ft (top of layer 8), respectively. Stresses in these plots have also been corrected by

factors of 1.65 and 0.60 to include the effects of reconstitution and to correct for differences in cyclic triaxial and cyclic simple shear data, as stated previously.

28. For the criterion of five cycles, Figure 21 shows that shear stresses of 872, 921, and 1170 psf will cause initial liquefaction, 10 percent peak-to-peak strain, and 20 percent peak-to-peak strain, respectively, in the sand layer immediately below the water table. Figure 22 shows that the criterion of five cycles gives shear stresses of 1715 and 2570 psf to cause initial liquefaction and 10 percent peak-to-peak strain, respectively, in the 14-ft sand layer located immediately below a depth of 45 ft. These values are tabulated in Table 3. Using the definition of factor of safety as previously stated, it is seen that factors of safety of 2.39 and 3.78 are obtained for these sand layers. These factors of safety are based on a failure criterion of 10 percent peak-to-peak strain. Thus, it is concluded that the medium to fine sand located between el 317 and el 350 will not fail if the left abutment is subjected to the design earthquake. This conclusion applies to the free-field condition, or at a location sufficiently distant from the riverbank such that no shear stresses exist on the horizontal planes prior to the earthquake.

Conclusions and Recommendations

29. The analyses reported herein are 1-D, and assume horizontal layers with zero shear stress on horizontal planes prior to the earthquake. Hence, they are not strictly applicable to other conditions, but serve as approximations from which more general conclusions may be drawn. The conclusions reported apply only to the cases studied. The input parameters were based largely on past WES experience; therefore, the results are, to some extent, qualitative. From these analyses, it can be concluded that the top sand layer downstream of the rock berm at the Newburgh site will liquefy if the structure is subjected to the assumed design earthquake. All other materials analyzed in these analyses are shown to be safe against 10 percent peak-to-peak strain.

30. It is recommended that a study be initiated to determine the cost and benefits of (a) removing the top 20 ft of sand below and downstream of the toe of the rock berm and replacing this material with nonliquefiable material (i.e., rock), and (b) extending the existing rock berm in the downstream direction. It is believed that such modifications might render the structure stable under earthquake excitation.

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Table 1

Analysis of Response of Case A

NEWBURGH
CASE A W/O ROCK BERM
G = 100 % G

LAYER	THICKNESS, FT	DEPTH TO TOP OF LAYER, FT	MAX SHEAR STRESS AT TOP OF LAYER, PSF	AVG SHEAR STRESS* AT TOP OF LAYER, IF N_{eq} = 5 CYCLES, PSF	UNIFORM CYCLIC SHEAR STRESS APPLIED FOR 5 CYCLES IN LAB CAUSES: INIT. LIQ PSF	10 % P-P STRAIN PSF	20 % P-P STRAIN PSF	FACTOR OF SAFETY $\left(\frac{\tau_{UNIF\ LAB}}{\tau_{AVG\ EQ}}\right)$ FOR 10 % ϵ
1	10	0						
2	10	10	555	319	161	170	216	0.53
3	9	20	929					
4	9	29	1119	675	720	1095		1.62

*THIS NUMBER IS BASED ON MEAN LAB DATA AND COUNTS THE 1ST PEAK AFTER A ZERO CROSSING.

Table 2

Analysis of Response of

Case B

NEWBURGH
CASE B W/ROCK BERM
G = 100 % G

LAYER	THICKNESS, FT	DEPTH TO TOP OF LAYER, FT	MAX SHEAR STRESS AT TOP OF LAYER, PSF	AVG SHEAR STRESS* AT TOP OF LAYER, IF N_{eq} = 5 CYCLES, PSF	UNIFORM CYCLIC SHEAR STRESS APPLIED FOR 5 CYCLES IN LAB CAUSES			FACTOR OF SAFETY $\left(\frac{\tau_{UNIF\ LAB}}{\tau_{AVG\ EQ}}\right)$ FOR 10 % ε
					INIT. LIQ PSF	10 % P-P STRAIN PSF	20 % P-P STRAIN PSF	
1	7	0						
2	19	7						
3	19	26						
4	19	45						
5	10	64						
6	10	74	1007	524	1240	1298	1658	2.47
7	9	84						
8	9	93	1031	653	2430	3670		5.63

*THIS NUMBER IS BASED ON MEAN LAB DATA AND COUNTS THE 1ST PEAK AFTER A ZERO CROSSING.

Table 3

Analysis of Response of

Case C

NEWBURGH

CASE C - LEFT ABUTMENT

G = 100 % G

LAYER	THICKNESS, FT	DEPTH TO TOP OF LAYER, FT	MAX SHEAR STRESS AT TOP OF LAYER, PSF	AVG SHEAR STRESS* AT TOP OF LAYER, IF N_{eq} = 5 CYCLES, PSF	UNIFORM CYCLIC SHEAR STRESS APPLIED FOR 5 CYCLES IN LAB CAUSES			FACTOR OF SAFETY $\left(\frac{\tau_{UNIF\ LAB}}{\tau_{AVG\ EQ}}\right)$ FOR 10 % ε
					INIT. LIQ PSF	10 % P-P STRAIN PSF	20 % P-P STRAIN PSF	
1	7	0						
2	6	7	204	115				
3	6	13	241	143				
4	7	19						
5	9.5	26	440	247				
6	9.5	35.5	619	386	872	921	1170	2.39
7	7	45	906	564				
8	7	52	1089	680	1715	2570		3.78
9	9.86	59	1419	848				
10	9.86	68.9	1654	994				
11	9.86	78.8	2127	1267				
12	9.86	88.7	2339	1414				
13	9.86	98.6	2782	1670				
14	9.86	108.5	2912	1781				
15	9.86	118.4	3255	1974				

*THIS NUMBER IS BASED ON MEAN LAB DATA AND COUNTS THE 1ST PEAK AFTER A ZERO CROSSING.

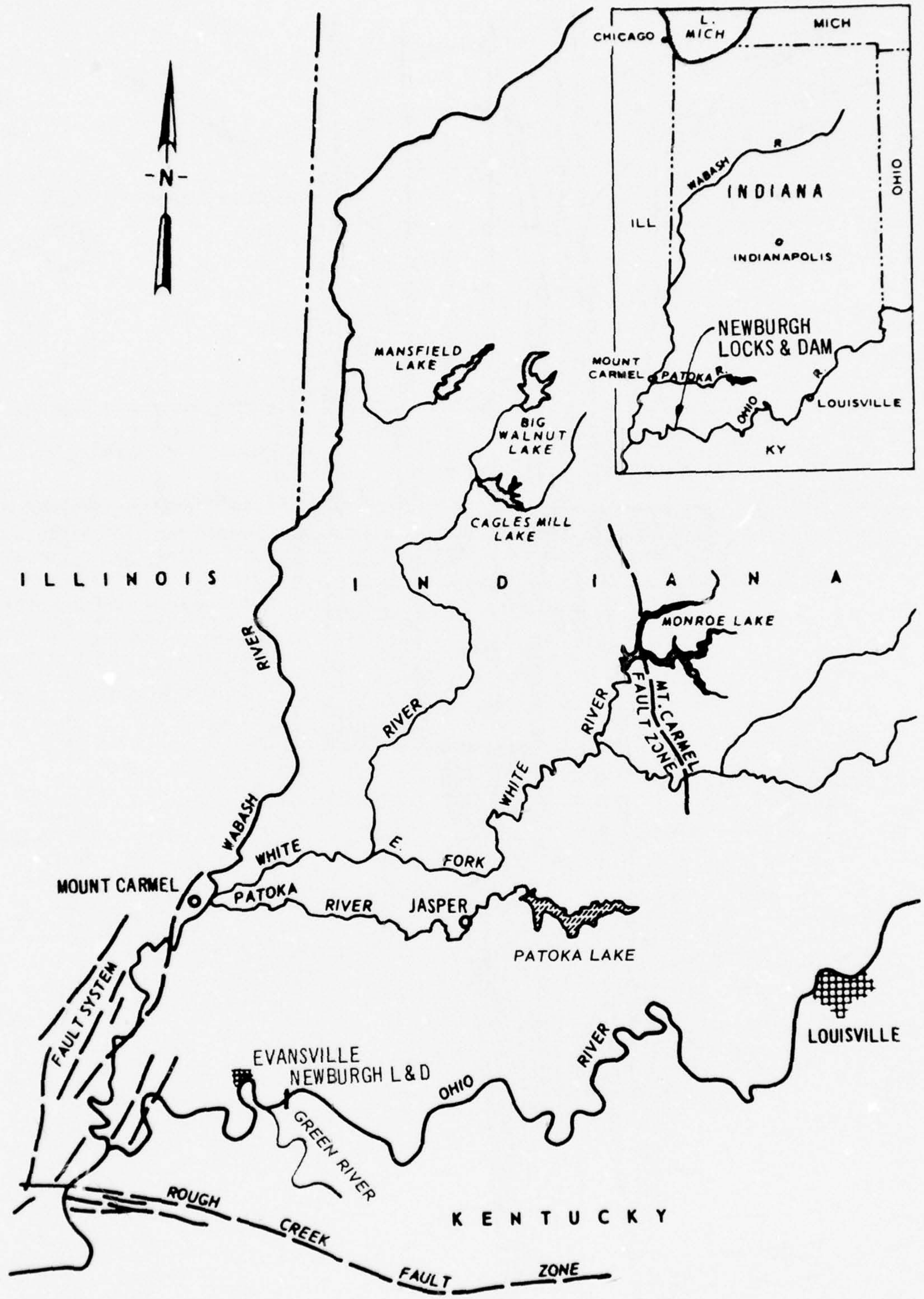


Figure 1. Location of Newburgh Locks and Dam

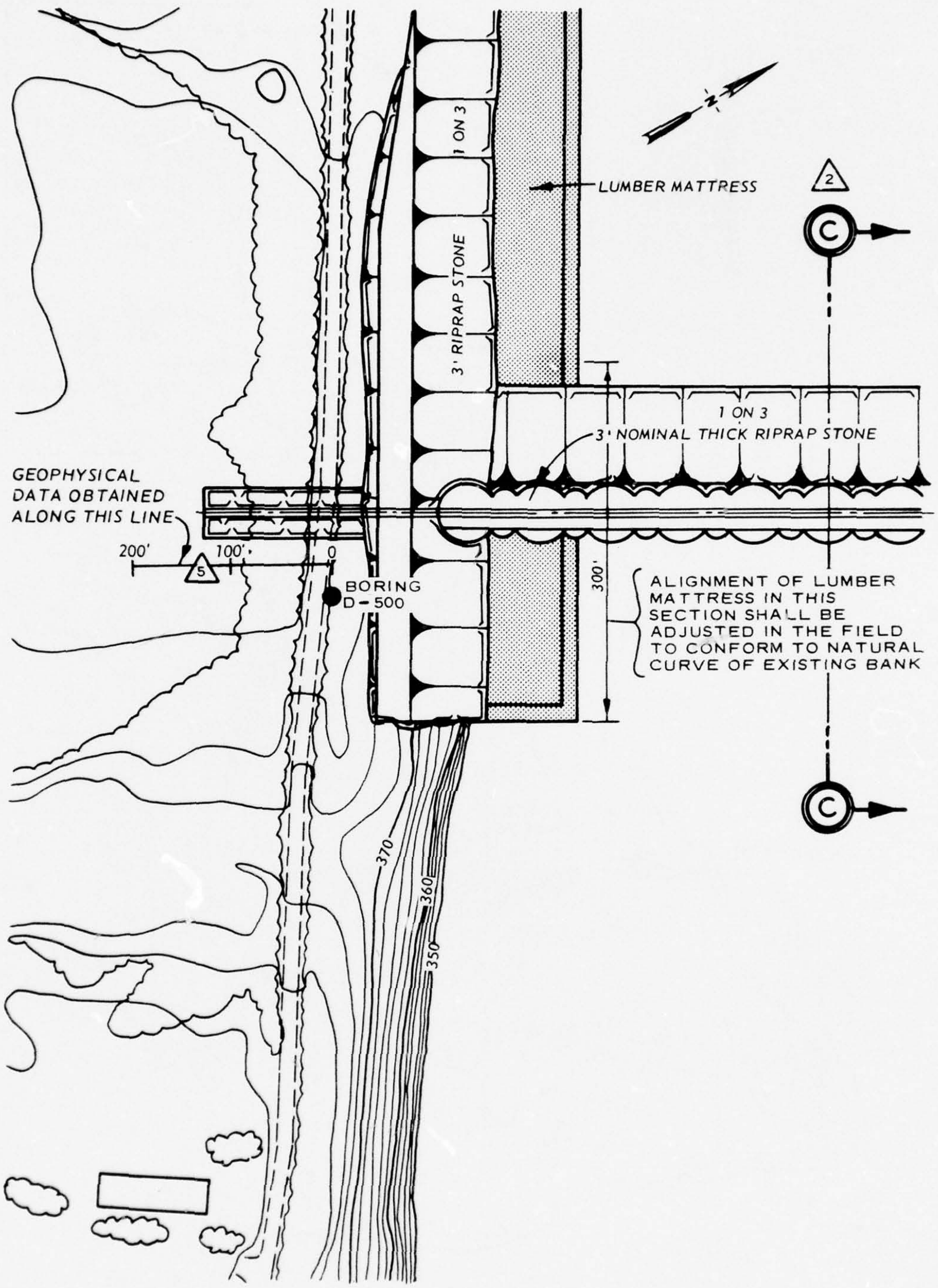
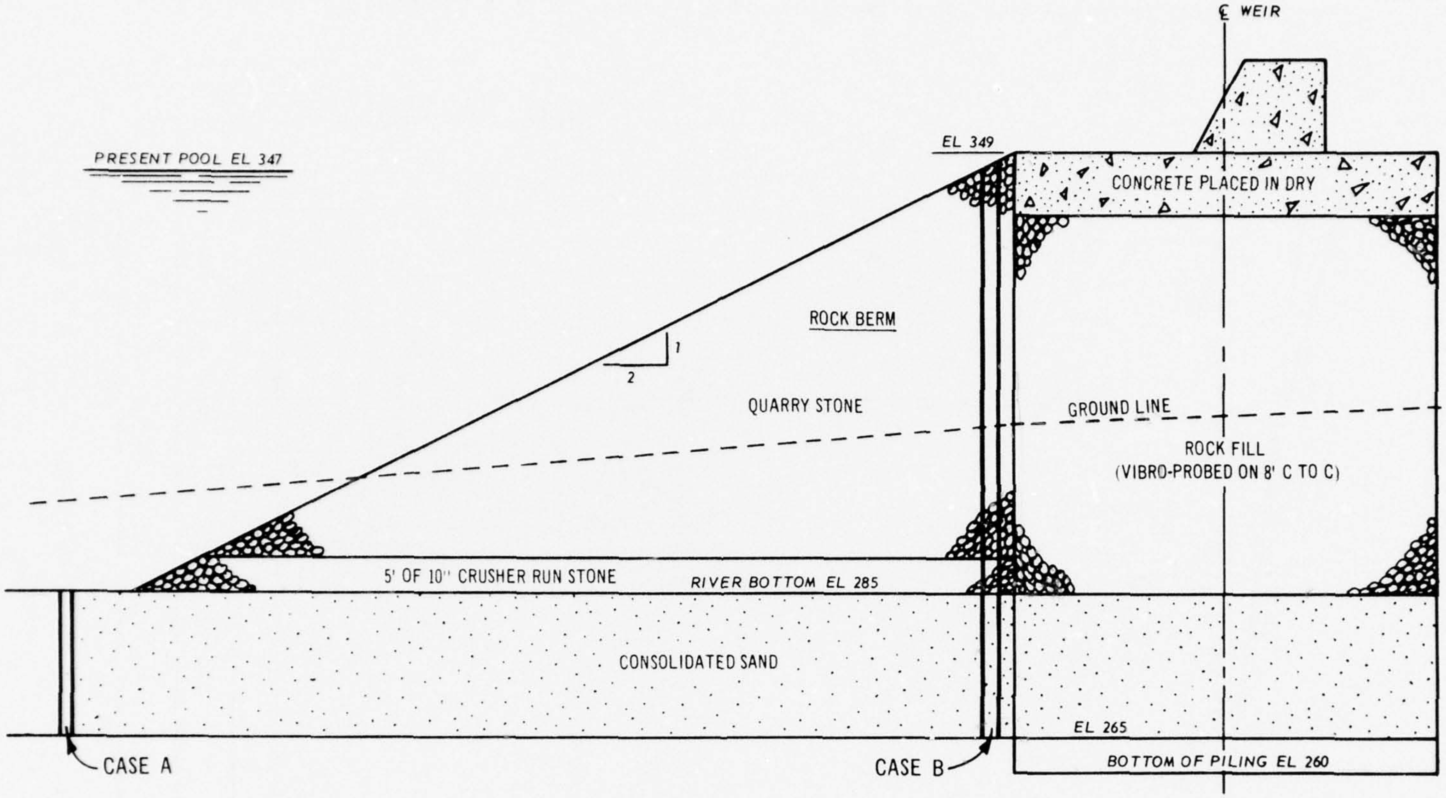


Figure 2. Plan view, Newburgh Locks and Dam
(Kentucky side only)

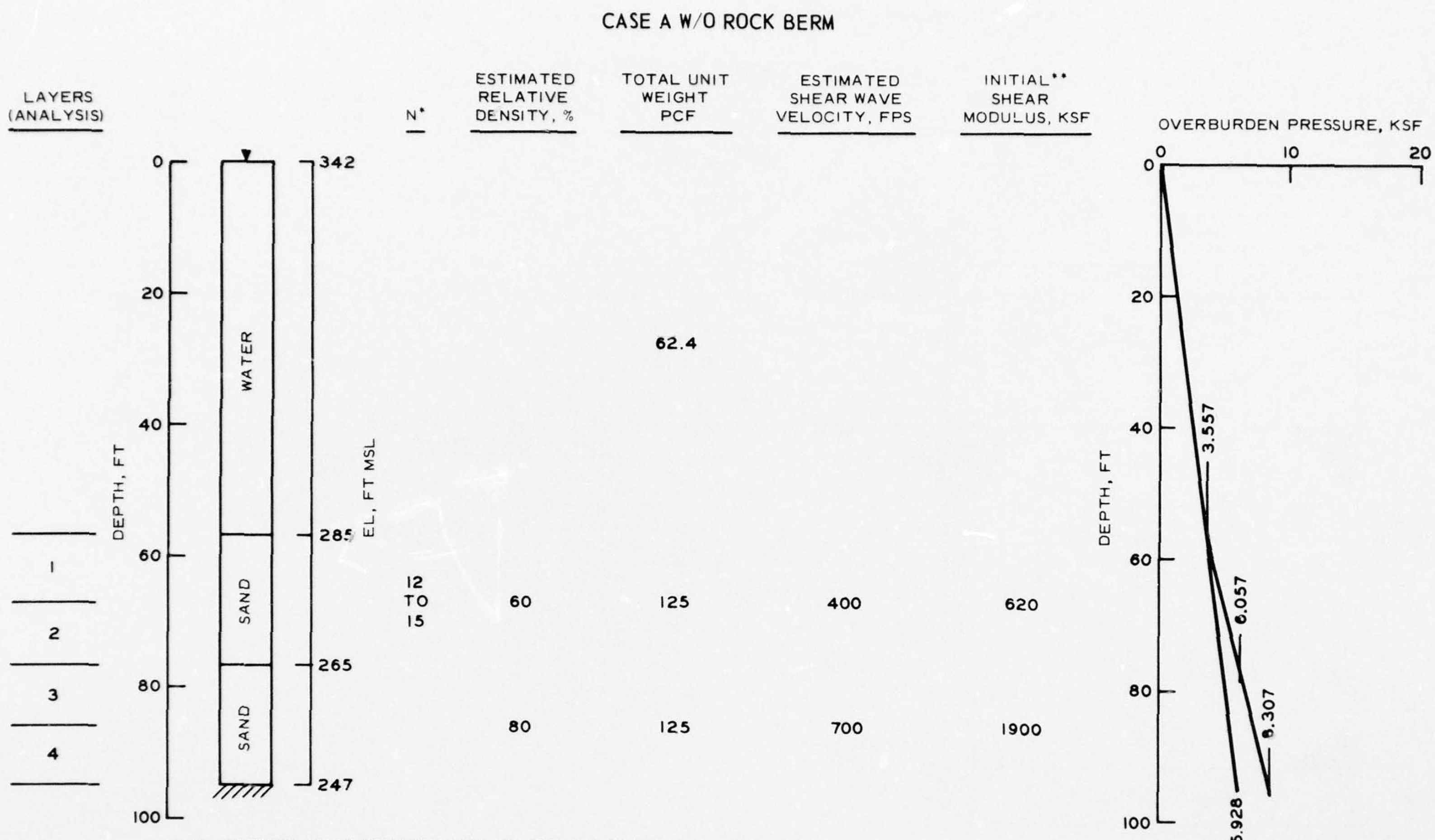


SECTION C – C

SCALE

10 0 10 20 30 FT

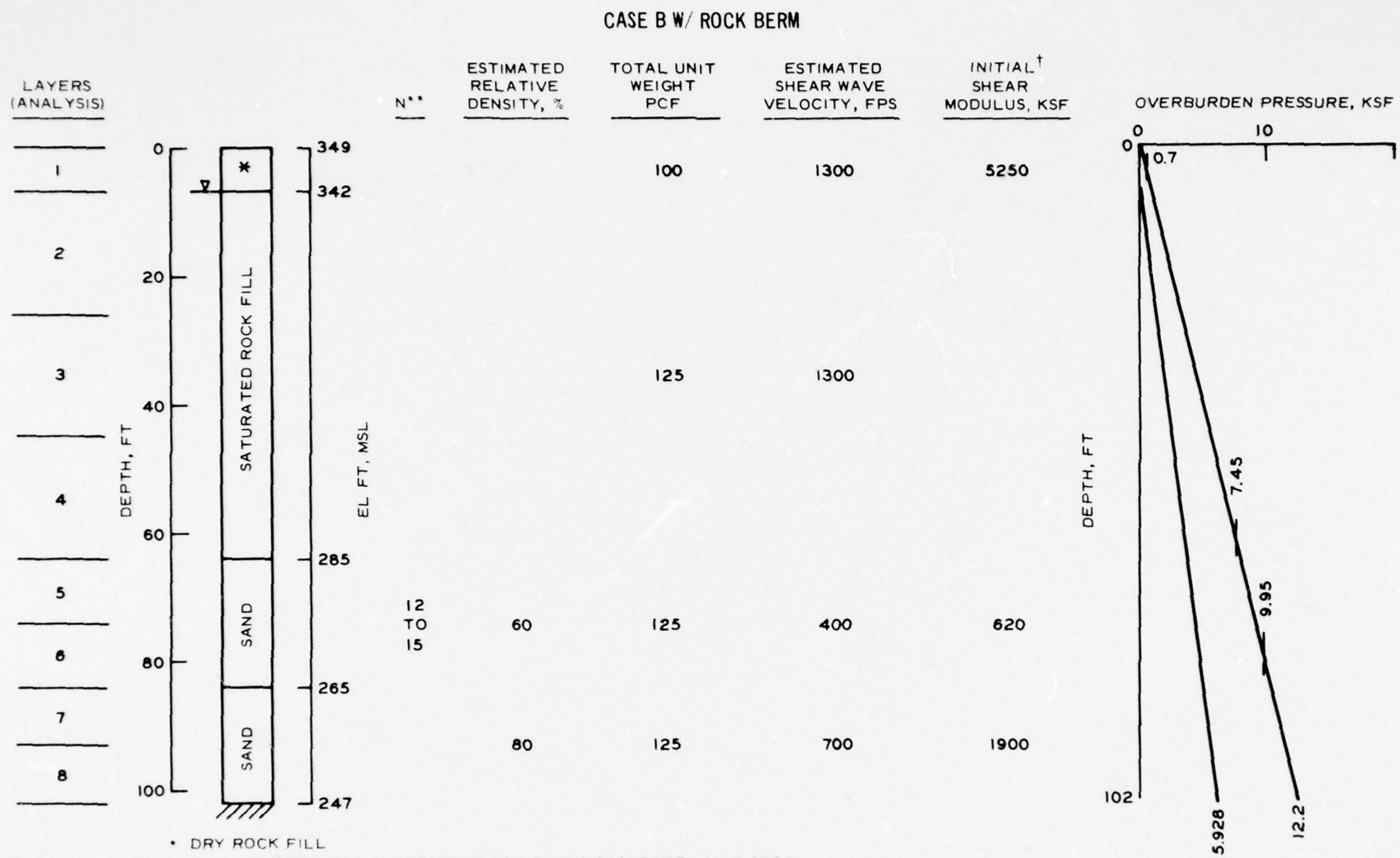
Figure 3. Cross section C-C, Newburgh Locks and Dam



* N = NUMBER OF BLOWS REQUIRED TO DRIVE A STANDARD SPLIT SPOON THE LAST 12 IN. ON AN 18-IN. DRIVE, USING A 140-LB HAMMER FALLING 30 IN.

** UNIFORM W/DEPTH

Figure 4. Idealized soil profile and properties used for 1-D analysis, Case A

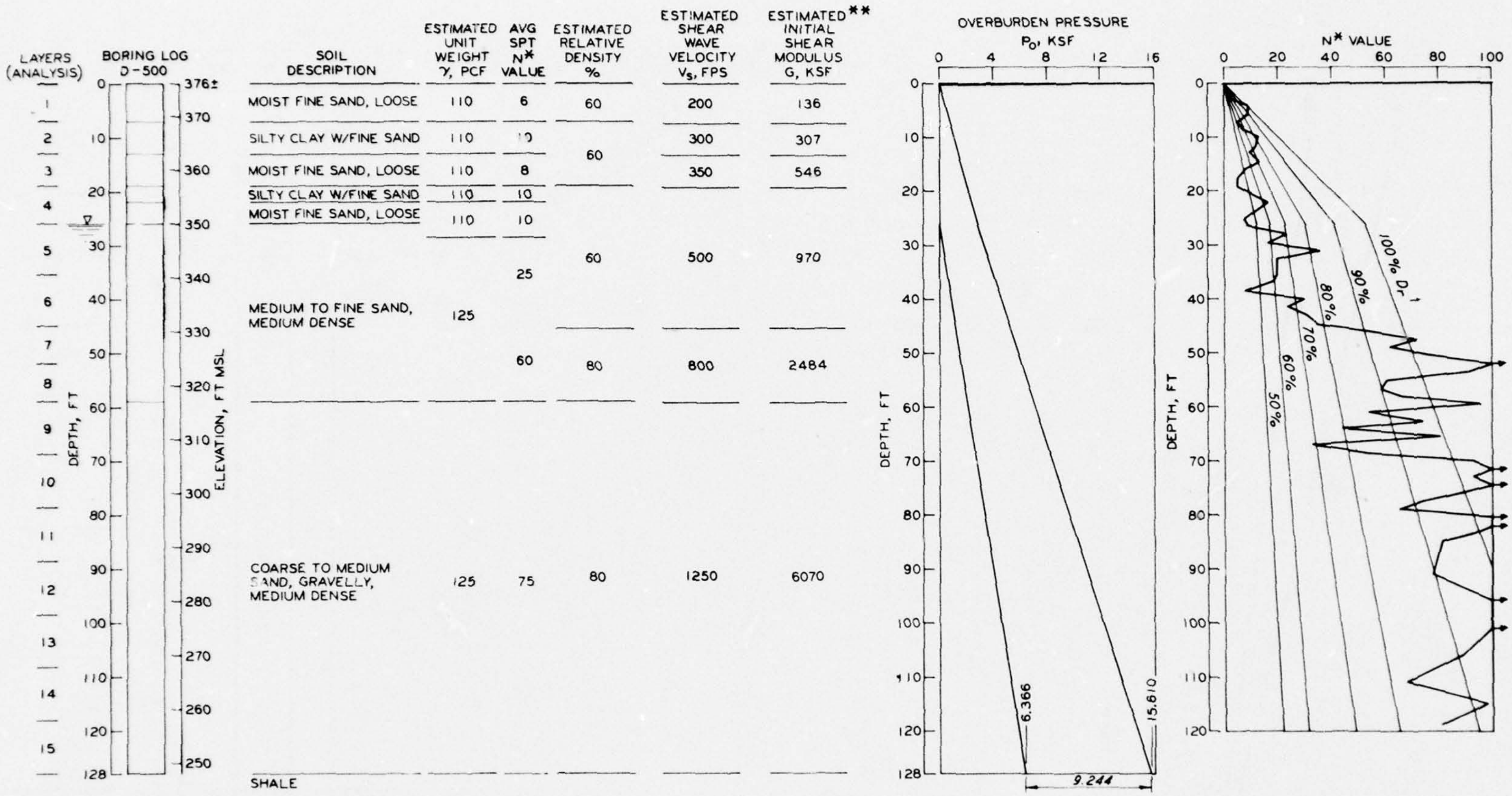


* DRY ROCK FILL

** N = NUMBER OF BLOWS REQUIRED TO DRIVE A STANDARD SPLIT SPOON THE LAST 12 IN. ON AN 18-IN. DRIVE, USING A 140-LB HAMMER FALLING 30 IN.

† UNIFORM W/DEPTH

Figure 5. Idealized soil profile and properties used for 1-D analysis, Case B



* N=NUMBER OF BLOWS REQUIRED TO DRIVE A STANDARD SPLIT SPOON THE LAST 12 IN. ON AN 18-IN. DRIVE, USING A 140-LB HAMMER FALLING 30 IN.
** UNIFORM W/DEPTH
† AFTER GIBBS & HOLTZ[12]

Figure 6. Idealized soil profile and properties used for 1-D analysis, Case C

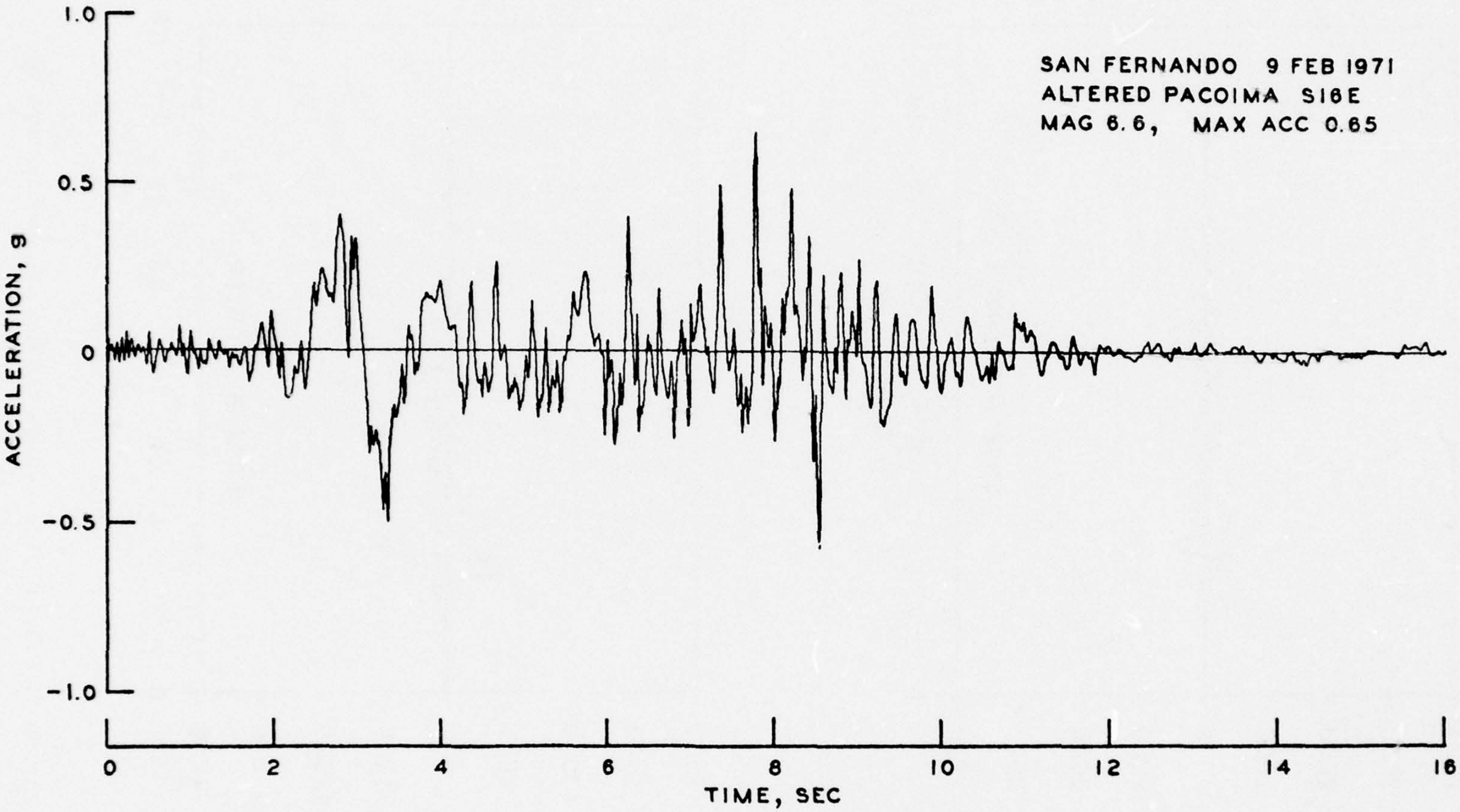
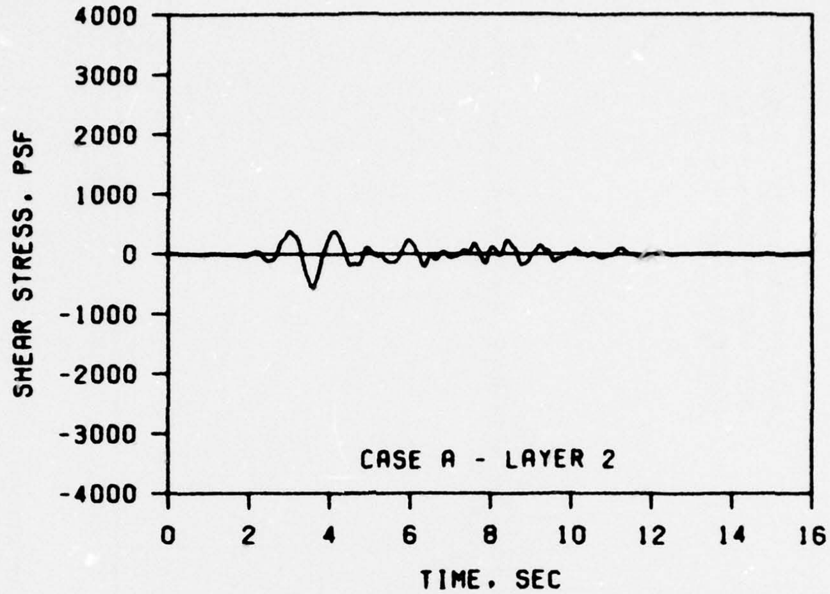


Figure 7. Altered Pacoima record used in the analysis of Lopez Dam[5]



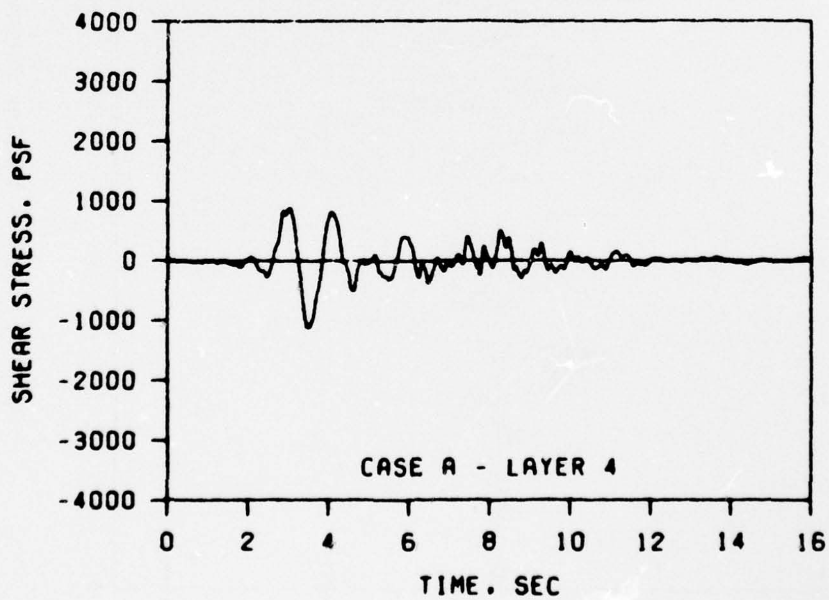


Figure 8. Shear stress-time histories for the tops of layers 2 and 4, Case A

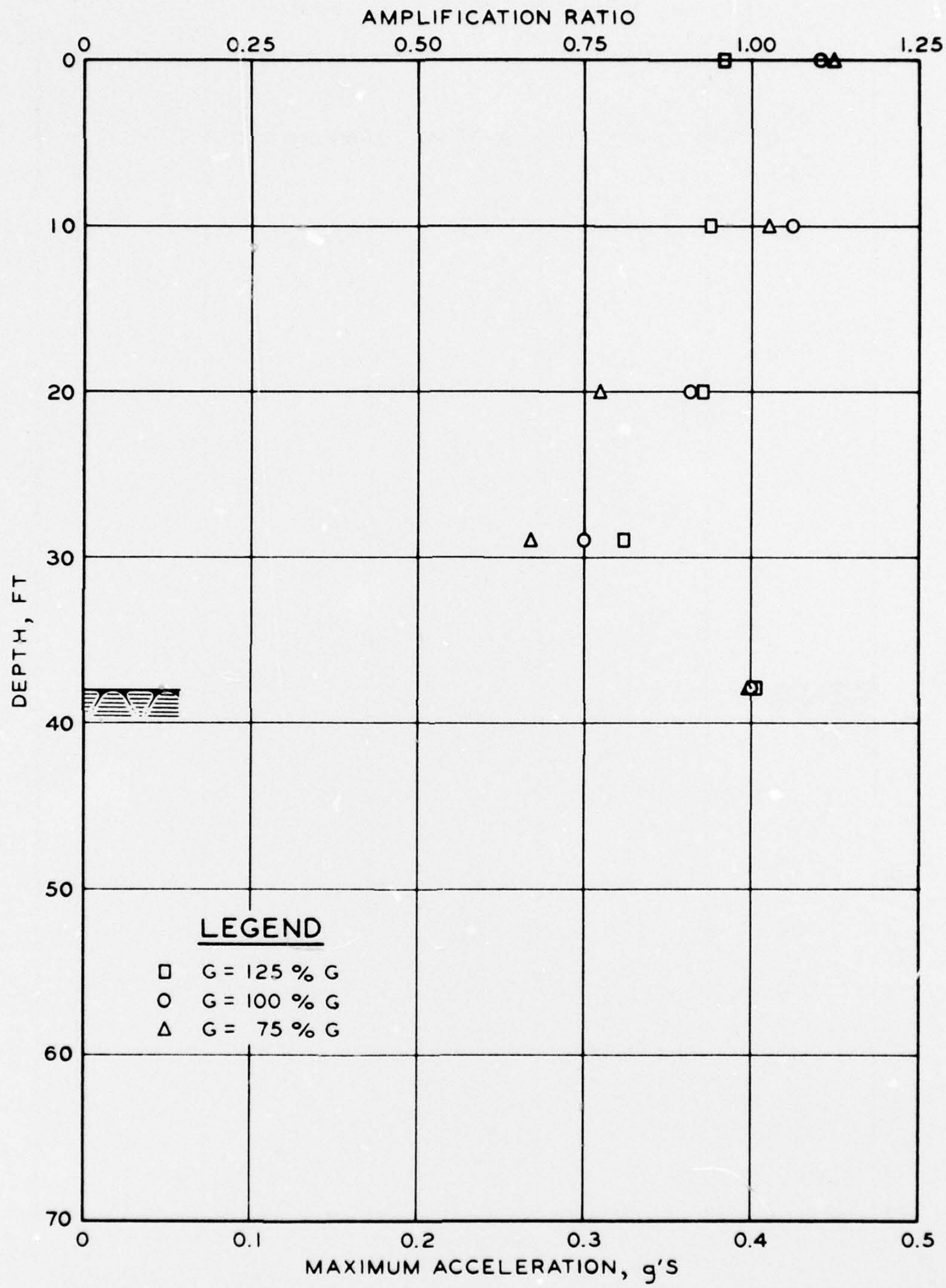


Figure 9. Maximum acceleration and maximum amplification ratio versus depth, Case A

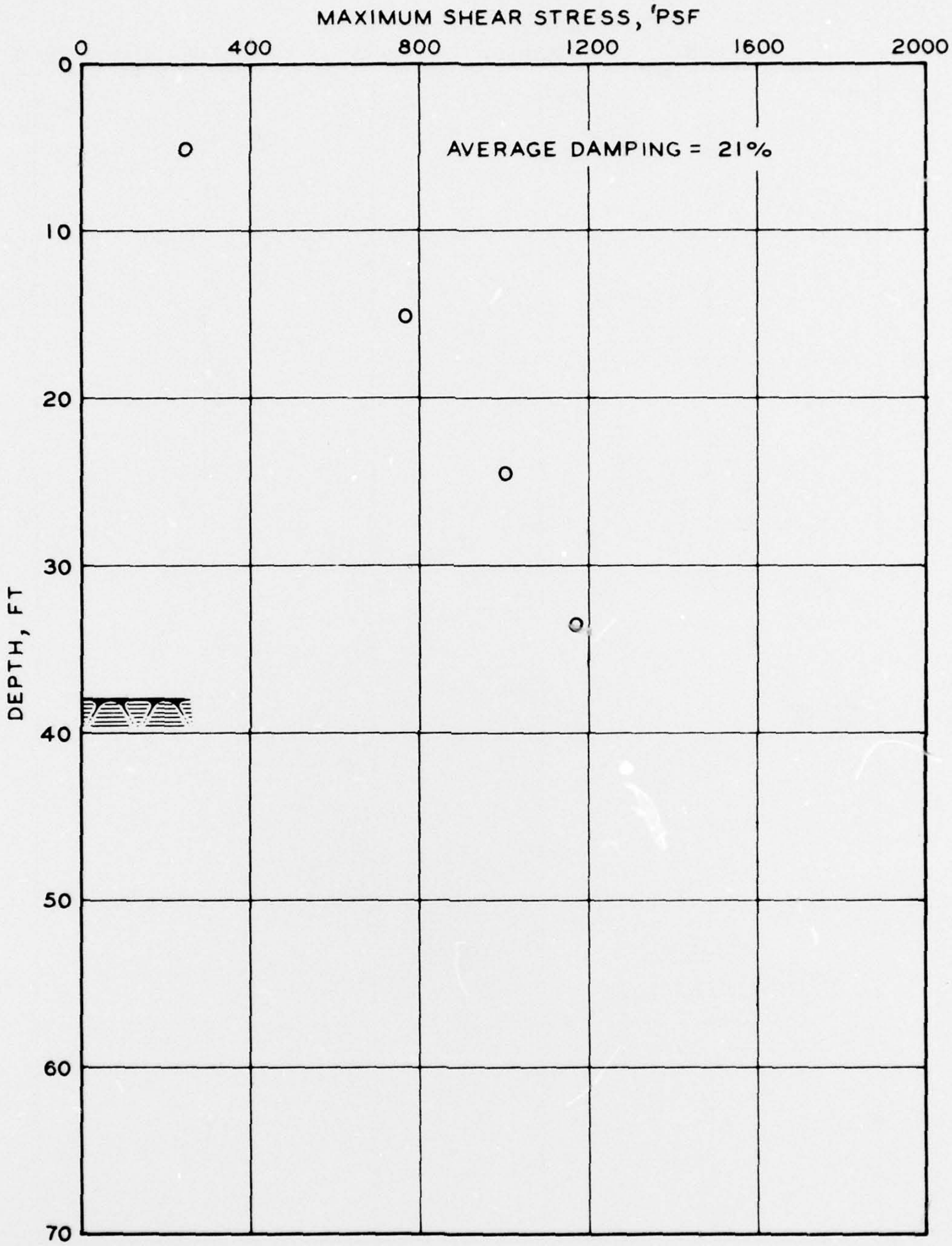


Figure 10. Maximum shear stress versus depth,
Case A

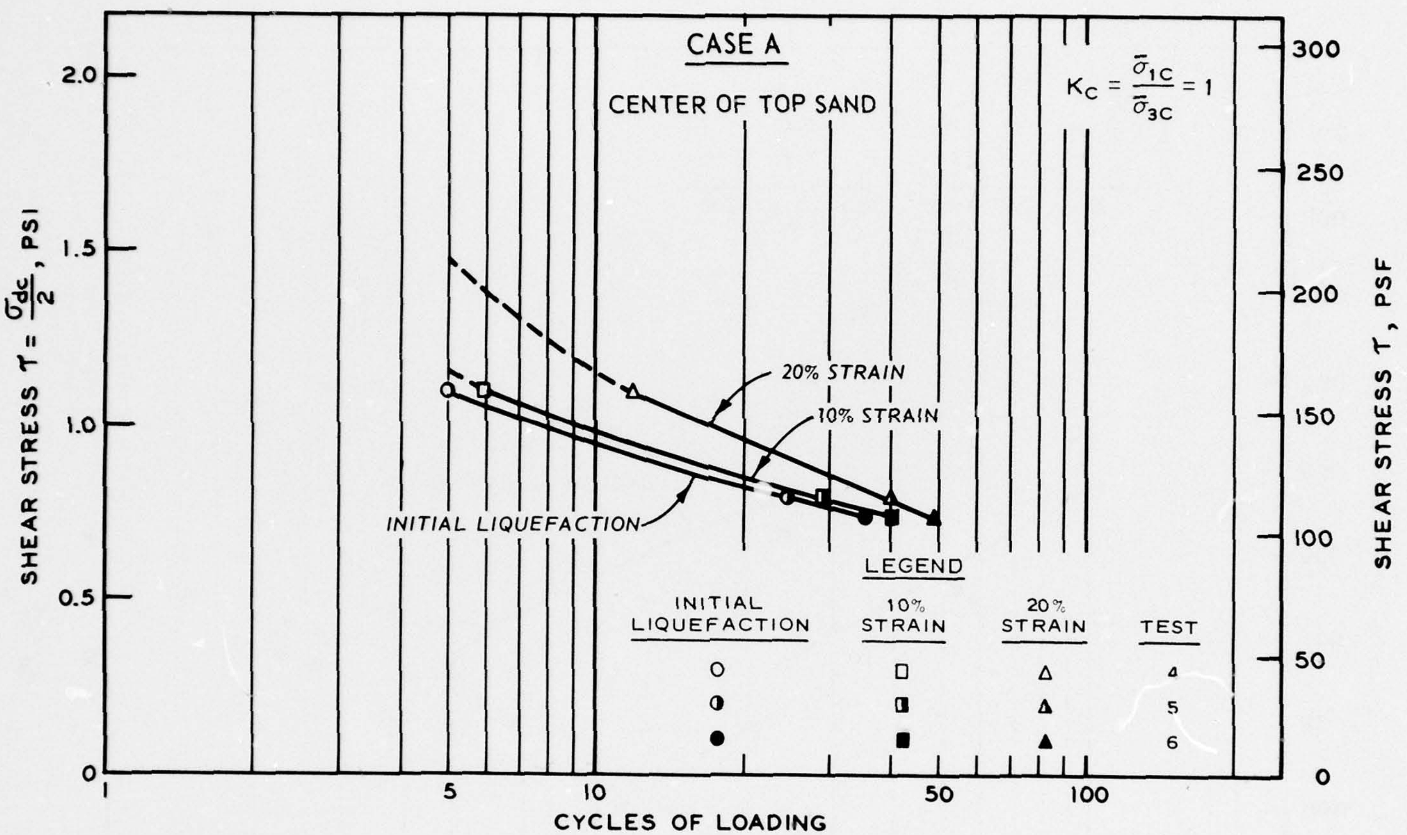


Figure 11. Cyclic triaxial tests of material at a relative density of 58.8%, Case A

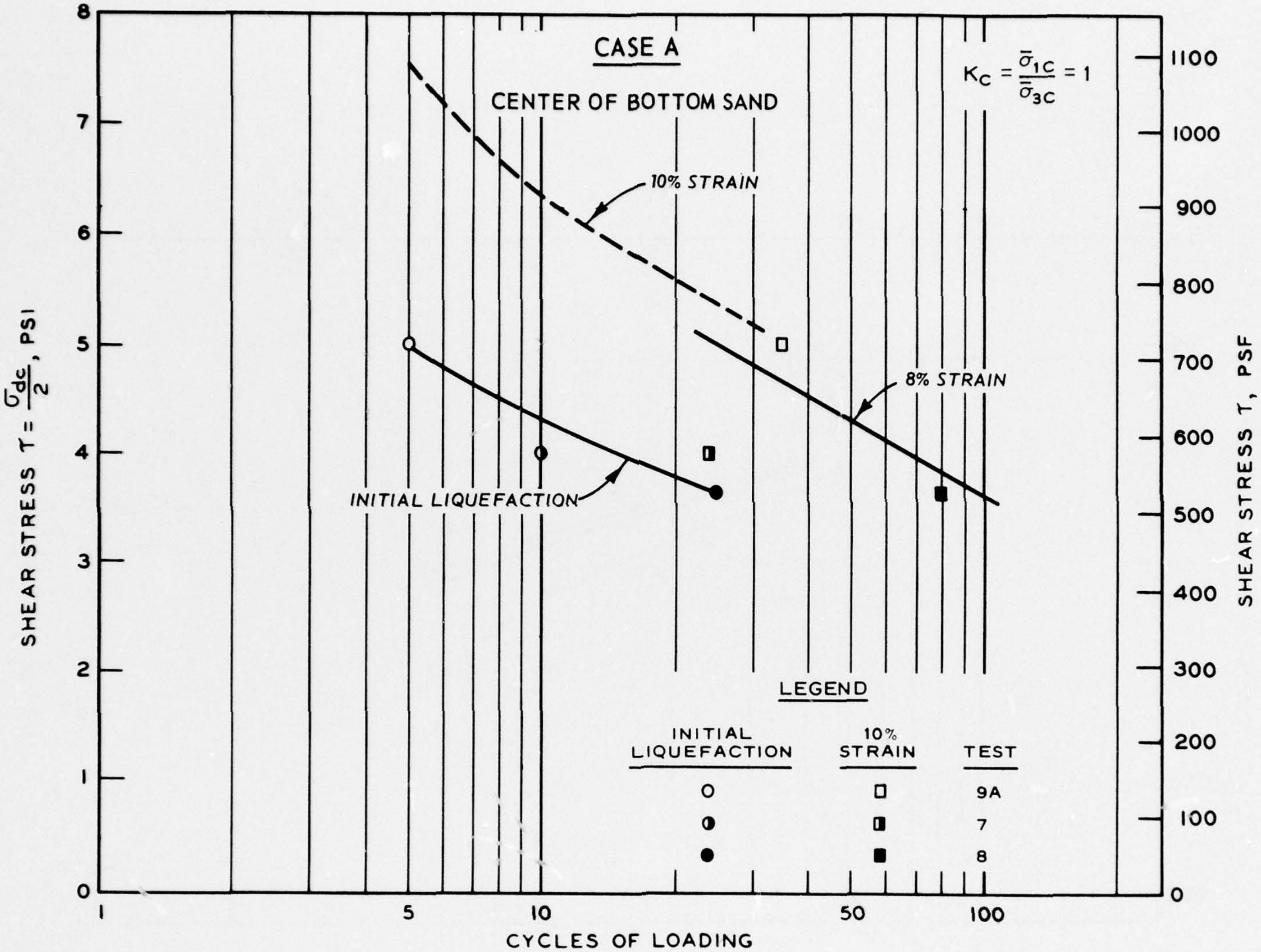
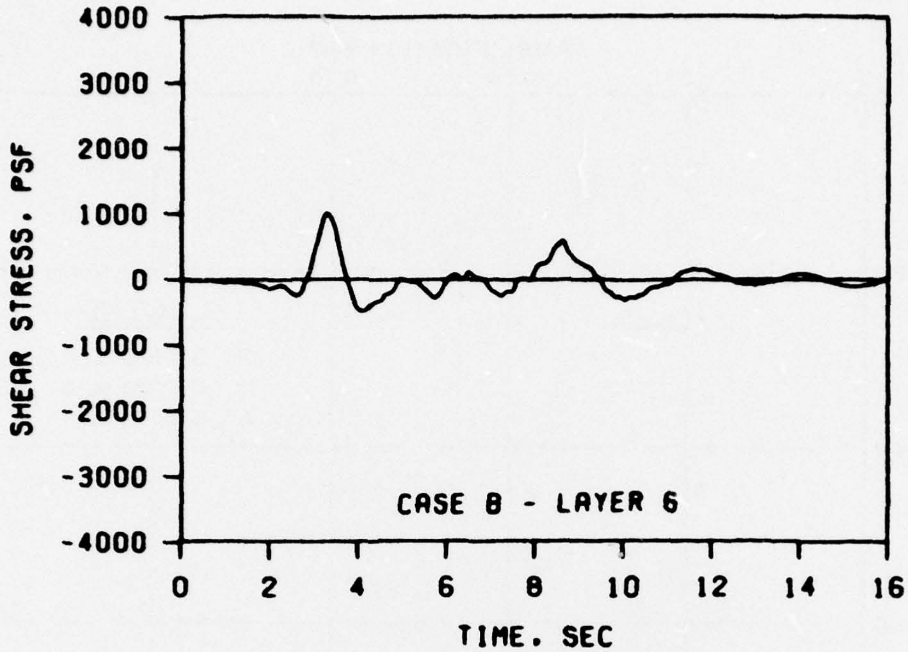


Figure 12. Cyclic triaxial tests of material at a relative density of 78.2%, Case A



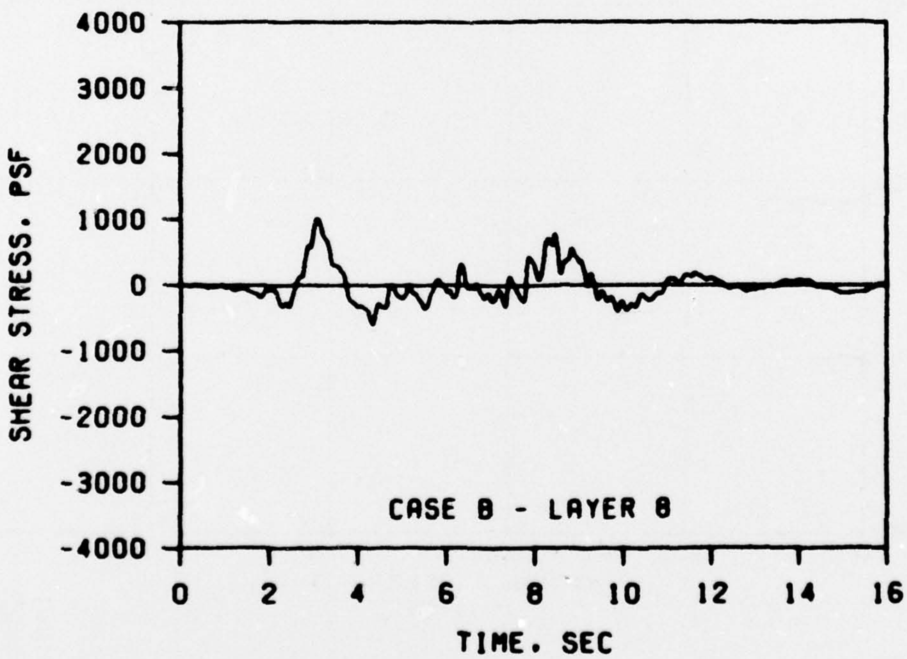


Figure 13. Shear stress-time histories for the tops of layers 6 and 8, Case B

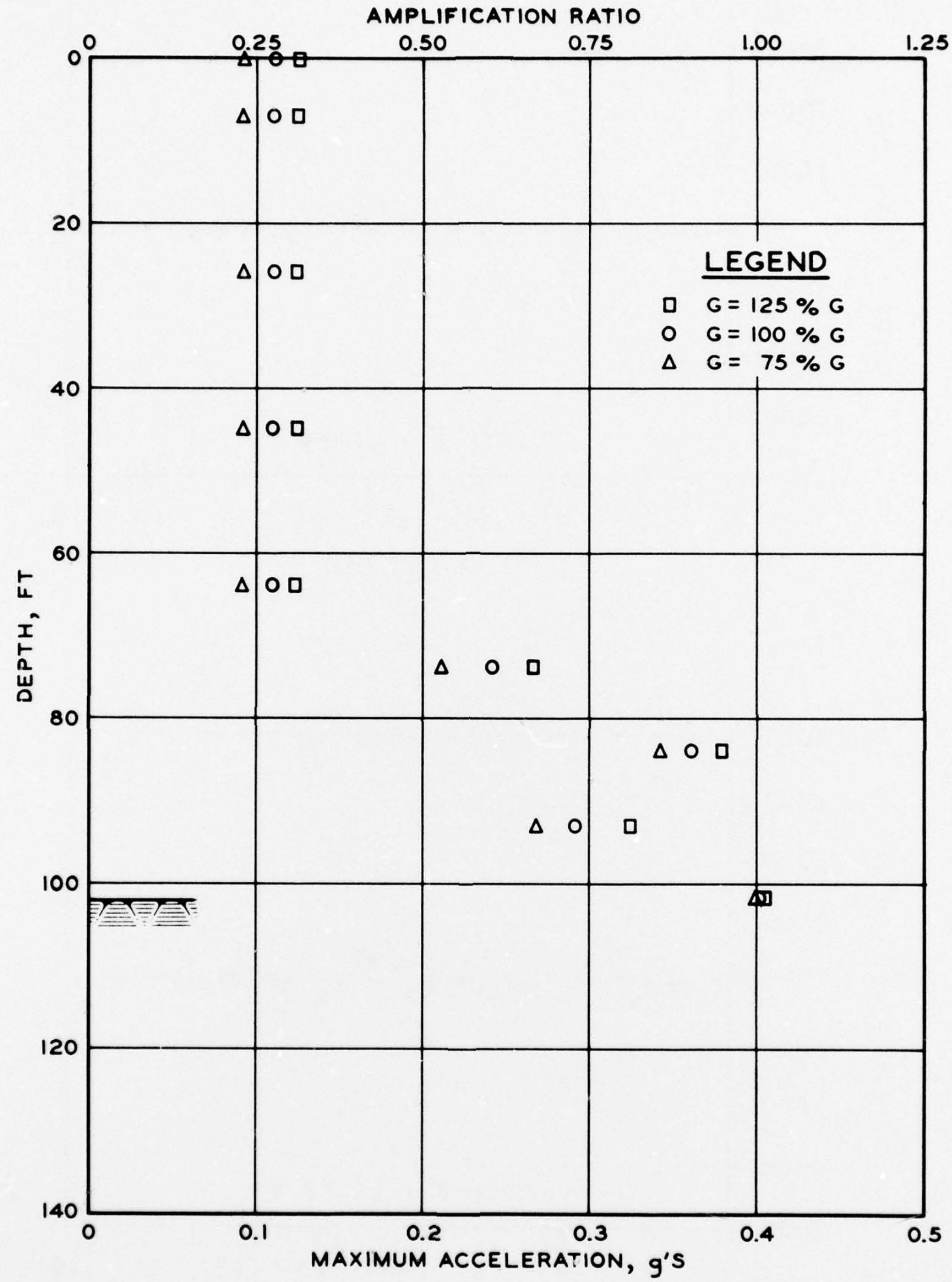


Figure 14. Maximum acceleration and maximum amplification ratio versus depth, Case B

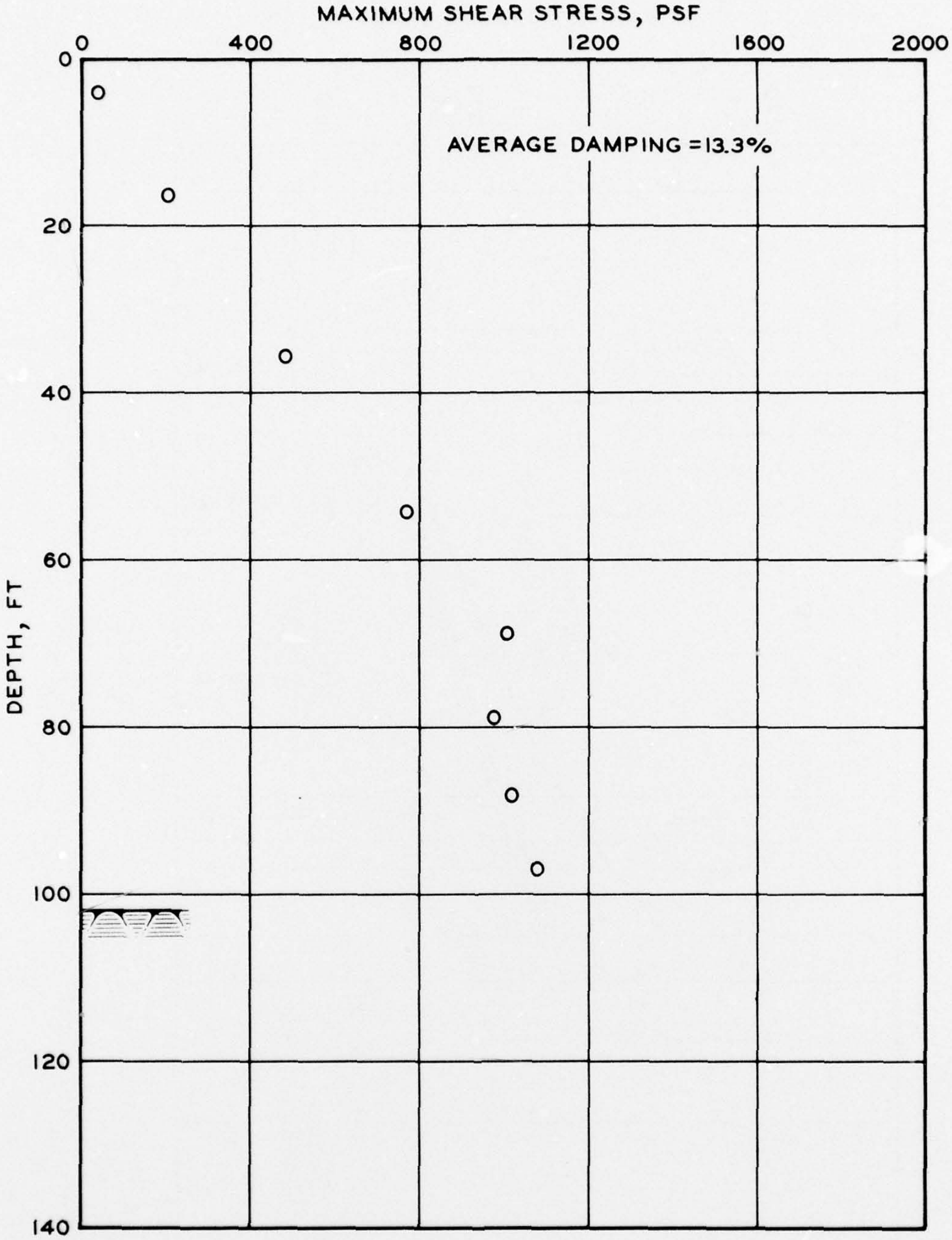


Figure 15. Maximum shear stress versus depth, Case B

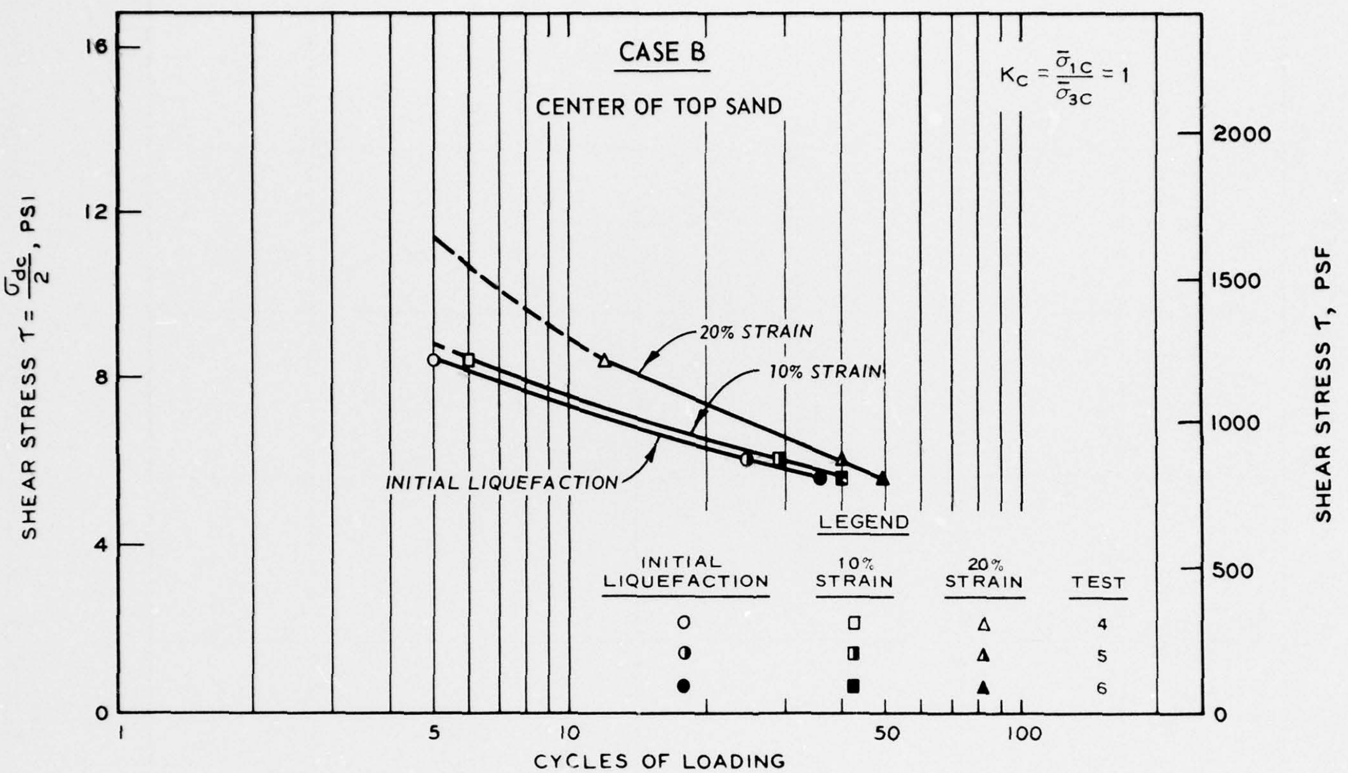


Figure 16. Cyclic triaxial tests of material at a relative density of 58.8%, Case B

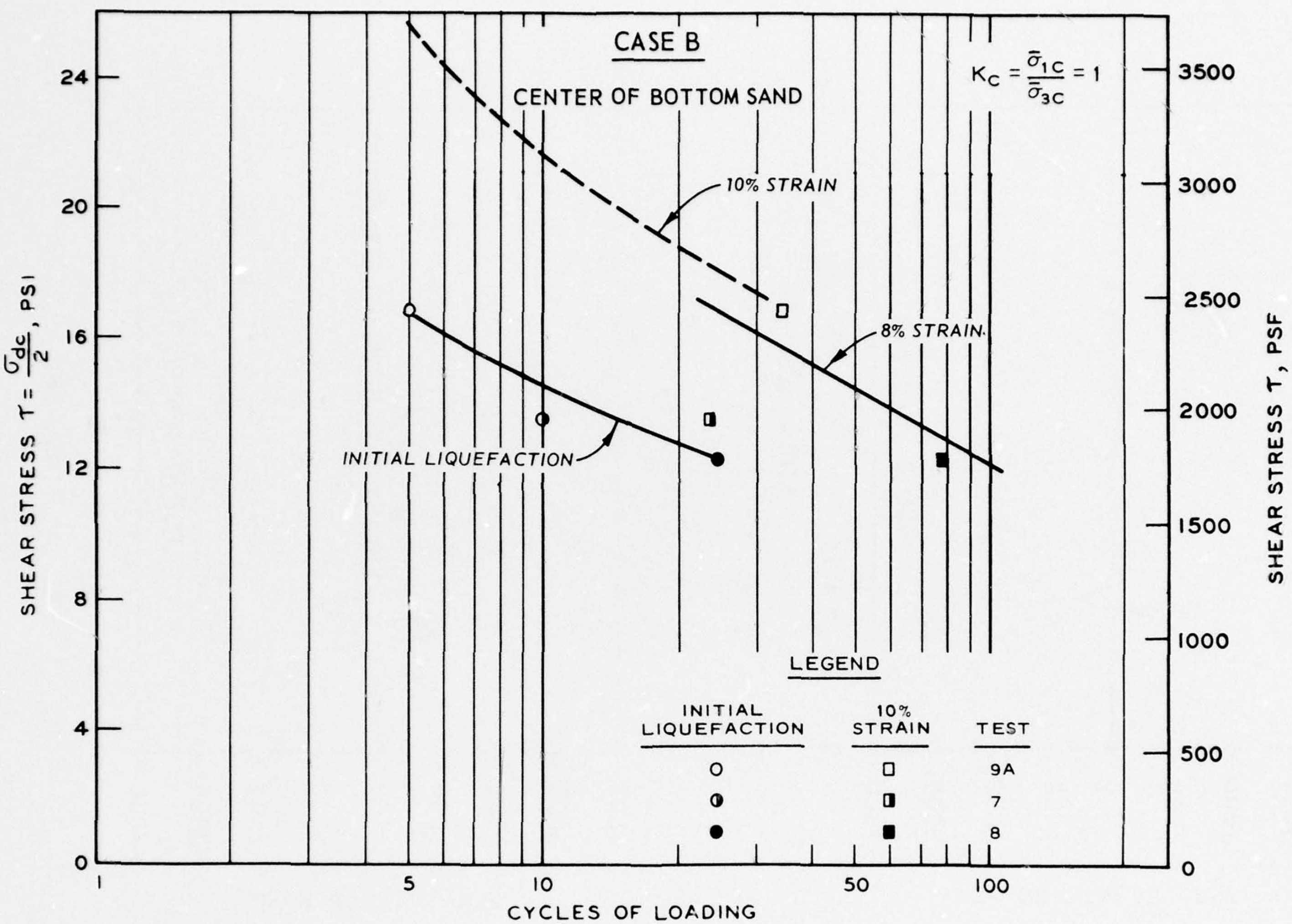
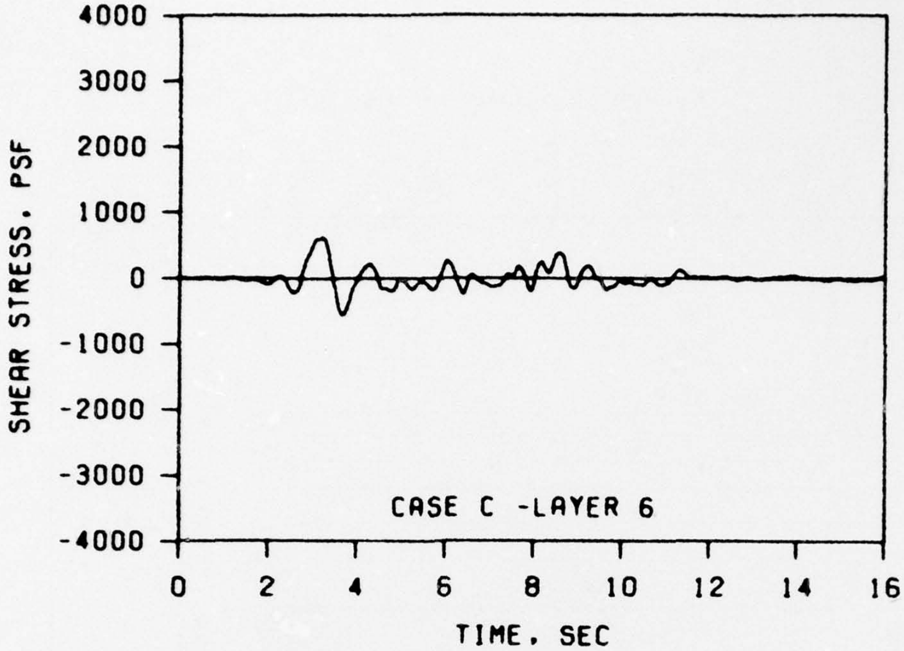


Figure 17. Cyclic triaxial tests of material at a relative density of 78.2%, Case B



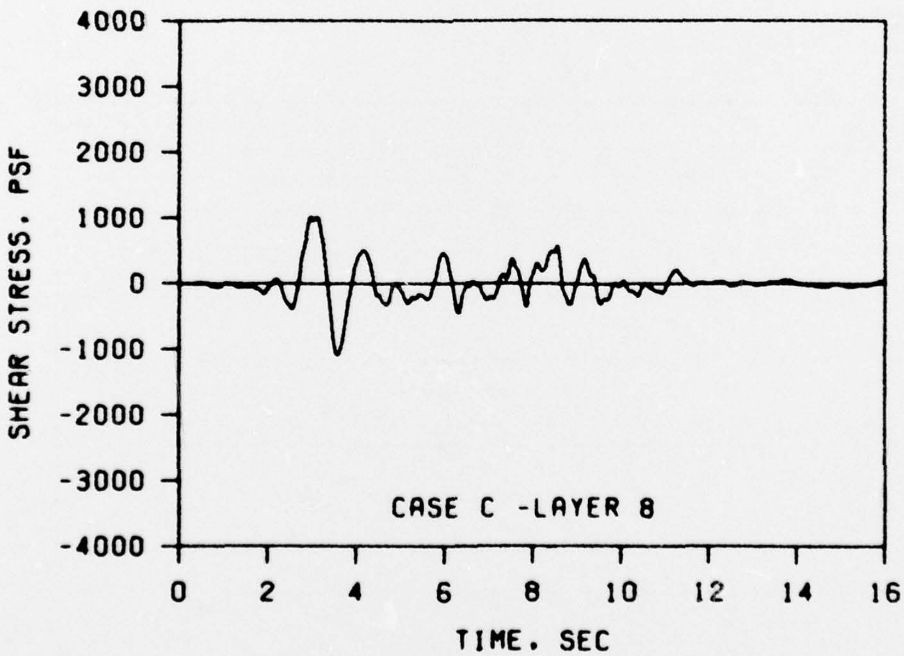


Figure 18. Shear stress-time histories for the tops of layers 6 and 8, Case C (Kentucky abutment)

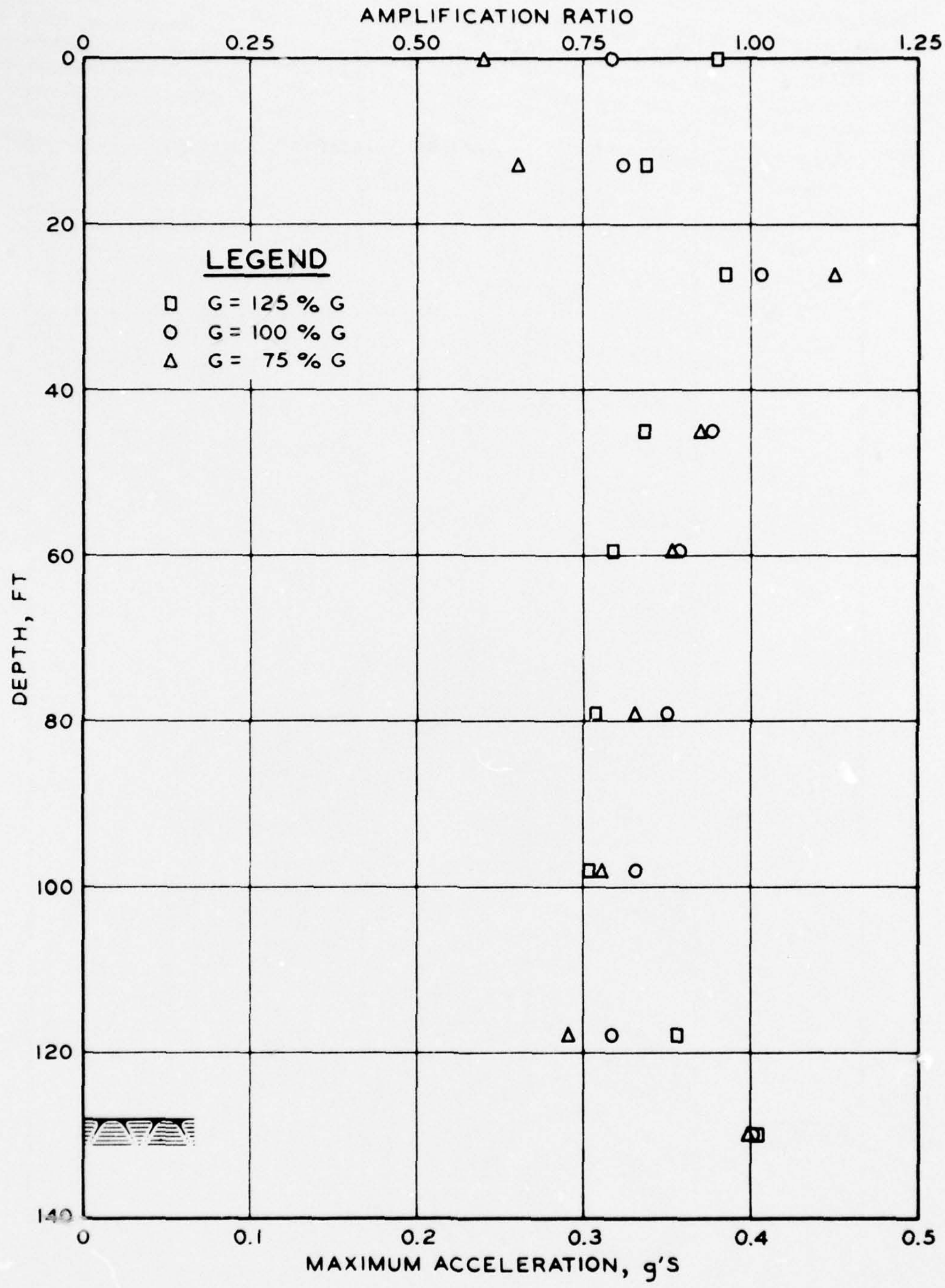


Figure 19. Maximum acceleration and maximum amplification ratio versus depth, Case C (Kentucky abutment)

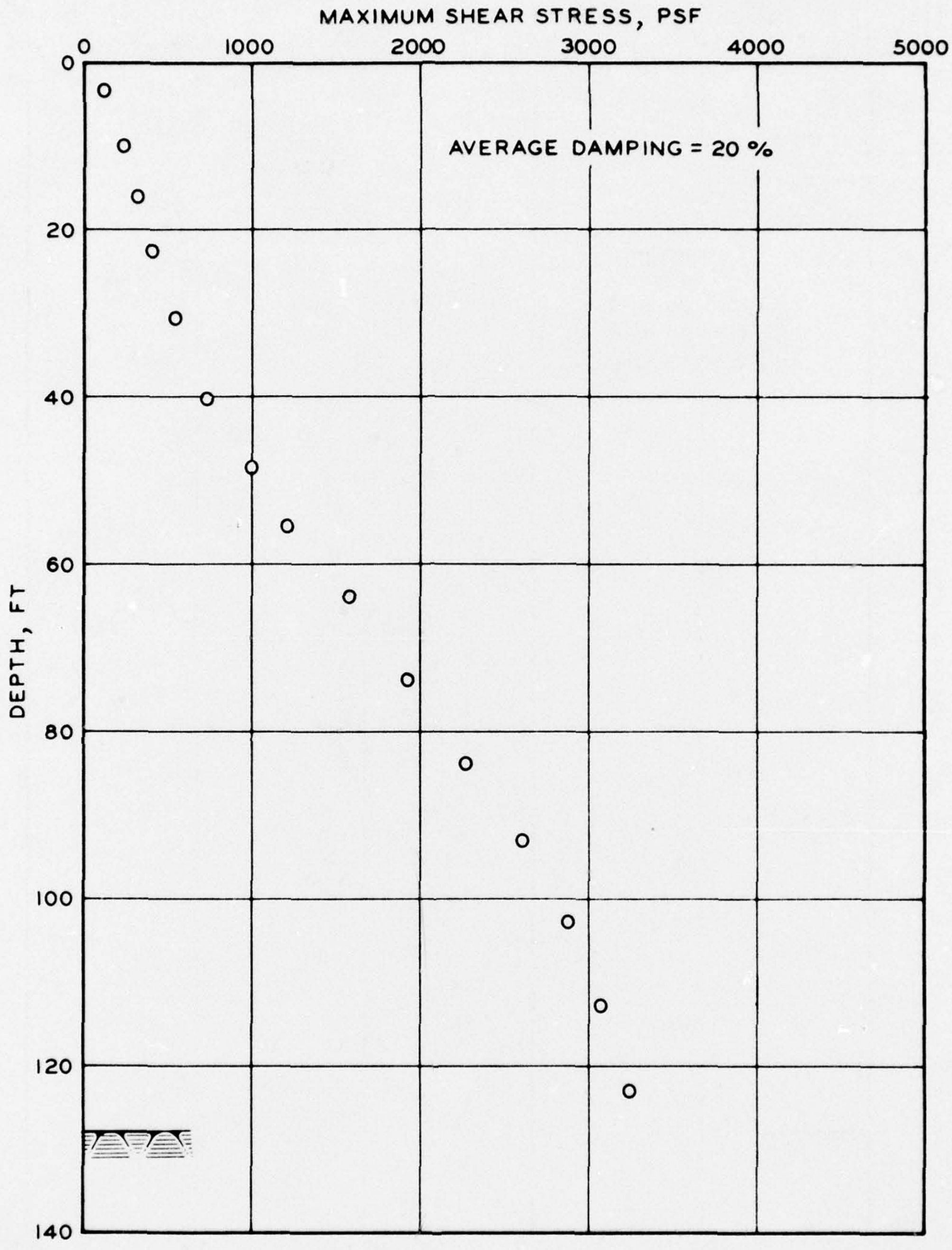


Figure 20. Maximum shear stress versus depth,
Case C (Kentucky abutment)

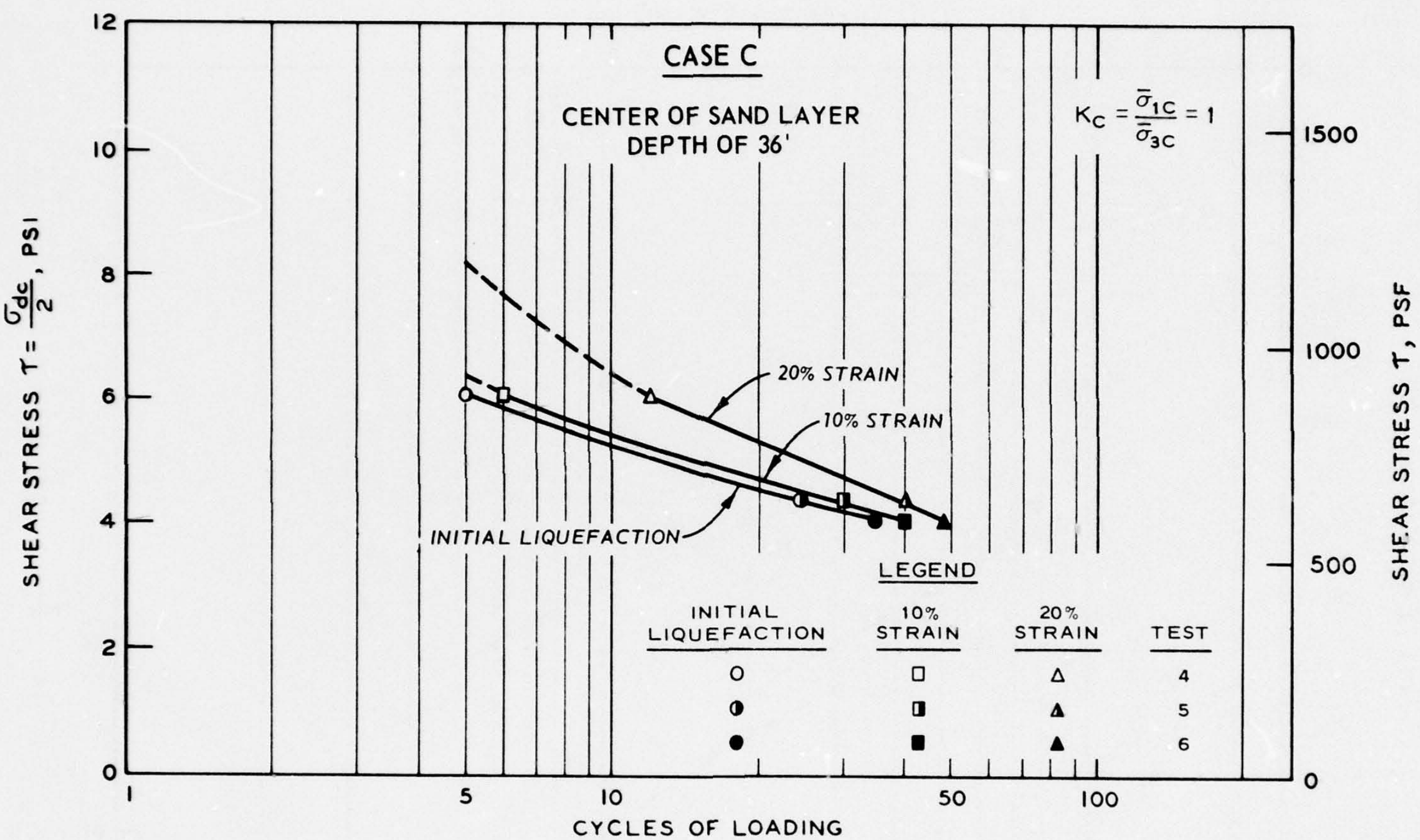


Figure 21. Cyclic triaxial tests of material at a relative density of 58.8%, Case C

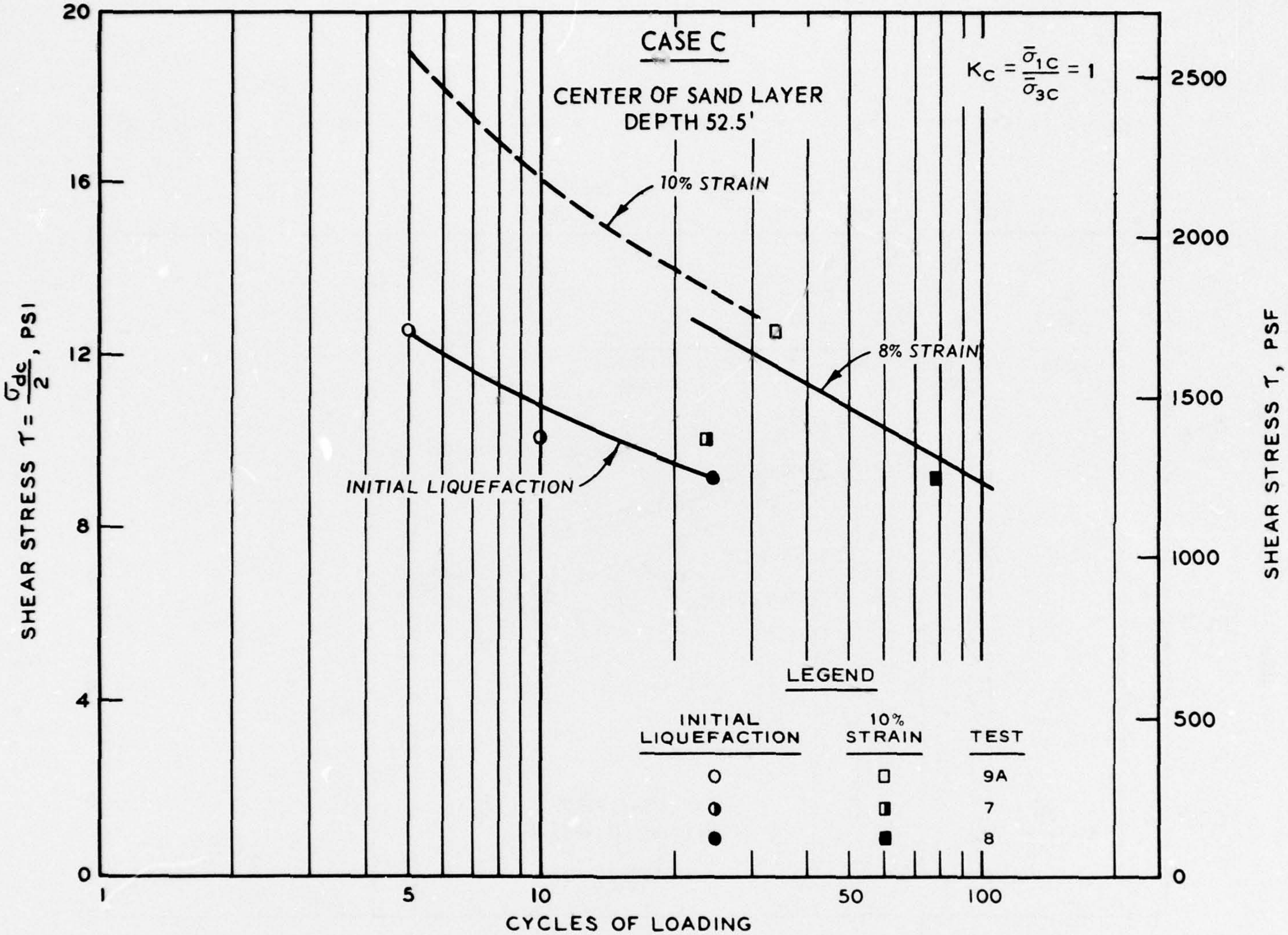


Figure 22. Cyclic triaxial tests of material at a relative density of 78.2%, Case C